सत्यमेव जयते

Ministry of Road Transport & Highways
Government of India

Capacity Development Project on Highways in Mountainous Regions

GUIDELINE FOR TUNNEL ON ARTERIAL ROADS IN HILL AREA

Published by :
Indian Roads Congress

On behalf of the Government of India,
Ministry of Road Transport & Highways
New Delhi

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Foreword

The road network plays a vital road in the overall development of a country by providing the access to employment, social, health and education. India has a vast area in hilly regions consisting of the Himalayan region from North to North-East, the Central Highlands of Aravali, Vindhya & Satpura ranges, the Sahyadri (Western ghats) and the Eastern ghats. Providing all weather connectivity road network in these hilly regions with extreme climatic conditions, difficult and hazardous terrain, topography and high altitude areas is a tough task. The Ministry of Road Transport & Highways has always taken steps towards the betterment of road transport throughout the country and tackling the problem of roads in hilly regions of the country is one such step.

This Guideline for Tunnel on Arterial Roads in Hill Area was prepared through the "Capacity Development Project on Highways in Mountainous Regions," bilaterally implemented by MoRTH and JICA. The project started in 2016 and prepared five guidelines that are essential for sustainable development of hill roads in India. Among the five guidelines, this Guideline for Tunnel on Arterial Roads in Hill Area focuses on the following technical topics:

- Evaluation of the ground for the design of tunnel supports by NATM, with elastic speed of the ground obtained through seismic exploration and rock/soil properties obtained through core boring.
- Basic knowledge for the design and construction of tunnel with NATM theory.
- Special and supplemental works against fragile / weak ground condition, such as fractured / sheared zones of bedrock, uneven earth pressure, unfavorable gas leakage, groundwater burst, extremely strong earth pressure which may lead rock burst, swelling rock and/or release of earth pressure by excavation which may cause squeezing of cross sections and/or pushing back of faces.
- Methods for quality control and monitoring during tunnel construction.
- Safety precautions for tunnels in view of design and construction.
- Tunnel operation facilities including ventilation system, tunnel lighting and tunnel electric room.
- Good practice in Japan in terms of mountain tunnel design and construction.

The Japanese team as well as the Indian team, especially the Experts from JICA, Counterpart Officials from MoRTH, NHAI & NHIDCL, the Project Director and the Project Team, are to be complimented for their commendable effort in producing these guidelines. Their contribution would go a long way towards the sustainable development of roads in the hilly regions of the country.

The 27th day of January 2022

(I. K. Pandey)
Director General (Road Development) & Special Secretary
Ministry of Road Transport & Highways
Government of India

PREFACE

India is a vast country with diverse geological, climatic regions. Roads are one of the important elements of transport infrastructure for land surface movement of passengers, goods, ammunition & army –intercity, intracity, rural areas and neighboring counties. Indian road network passes through different terrains including hills and mountains. Mountainous regions consist of Himalayan region from North to North-East, the Central Highlands of Aravalli, Vindhya and Satpura ranges, Western and Eastern Ghats.

These mountainous regions, generally, have extreme climatic condition, difficult and hazardous terrain, topography and vast high-altitude areas. Roads in mountainous areas are often composed of multiple civil engineering structures including bridges and tunnels, and proper maintenance is essential to maintain the road function as a network. The Hill regions including the road networks are affected more frequently by floods due to torrential rainfall, landslides, snow avalanche etc. which compel many sections of road network closed during rainy season and winter season. However, accountability for providing safe, reliable and user comfortable road network ultimately rests with the Government departments.

For hill roads IRC has published numerous guidelines, manuals, codes, etc viz IRC: 52, IRC:48, IRC: SP:91; IRC: SP:108: IRC: SP:113, IRC: 5; etc for guidance and adoption by highway professional & road departments.

Ministry of Road Transport & Highways (MORTH), took up the initiative to frame Guidelines for Planning, Slope protection, Mountain bridge, Mountain tunnel & Operation and maintenance of highways in mountainous regions through technical cooperation from Japan International Cooperation Agency (JICA) in 2016. JICA took site visits of hilly roads in India, collected site problem statement, interacted with Indian experts from MORTH, NHAI, NHIDCL, PWD, IRC, etc and also conducted seminars, webinars, meetings, etc before finalization of following 5 documents;

- Guidelines for Planning
- Guidelines for Slope protection and embankment with advanced technology
- Guidelines for Mountain bridge with advanced Technology

- Guidelines for Tunnel on Arterial Roads in Hill Areas
- Guidelines for Operation and maintenance

The Guidelines for Highways in Mountainous Regions, as prepared by JICA mainly highlights appropriate planning, design principles for promoting safety conscious planning and design of roads, embankment, slope & its operation & maintenance aspect and will go a long way in guiding the highway engineers and planners to make our, hill roads, tunnels, bridges, safer, comfortable and of international standards.

It is quite natural, this JICA publications cannot be expected to cover all the subjects and details of highway engineering and as such should not be considered as a substitute for the Standards and Specifications as well as Codes of Practice.

The intent of this publications is to provide guidance to the engineers with the critical aspects of development of Highways in Mountainous Regions. These guidelines are dynamic in nature and the users, based on their experience, give appropriate feedback so as to enable the Guidelines to be revised/improved from time to time.

(S K Nirmal)
Director General (Road Development) and Special Secretary
Ministry of Road Transport and Highways &
Secretary General, IRC

Dated July, 2022

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Table 9-1: Power receiving method 9-2

List of Abbreviations and Acronyms

ACGIH	American Conference of Governmental Industries Hygienists
BRO	Border Roads Organization
DMP	Deformation Monitoring Points
EDM	Electronic Distance Meter
JR	Japan Railway Company
JSCE	Japan Society of Civil Engineering
LPG	Liquefied Petroleum Gas
LVCT	Linear Variable Displacement Transformer
MPBX	Multiple Position Borehole Extensometers
NATM	New Austrian Tunneling Method
PPV	Peak Particle Velocity
SFRC	Steel Fiber Reinforced Shotcrete
SPBX	Single Position Borehole Extensometers
TBM	Tunnel Boring Machines

This "GUIDELINE FOR TUNNEL ON ARTERIAL ROADS IN HILL AREA" is written based on "Japan Society of Civil Engineers (2006); Standard Specifications for Tunneling – 2006".
Figures and Tables in this Guide are reproduced from the "Standard Specifications for Tunneling" unless otherwise stated

CHAPTER 1 PLANNING AND INVESTIGATION

1.1 Planning

1.1.1 Alignment and Tunnel Cross Section

(1) Horizontal Alignment of Tunnels

1) The alignment of tunnels shall be as straight as possible for ease of construction. If curved alignment is used, the radius shall be as large as possible.
The curve radius of tunnels shall be as large as possible, depending on the type and classification of the tunnel. In road tunnels, a slight curve at the tunnel exit may sometimes have the advantage of allowing drivers to adjust gradually to the brightness outside the tunnel.
Tunnels shall be situated in good topographic and geologic conditions and have enough overburden and lower ground level, considering the investigation results and monitoring to preserve the surrounding environment. Tunnel alignment shall be studied considering the location or construction conditions, such as the position of ventilation, emergency or working drift, outside construction facilities, and muck disposal area.

2) The tunnel portal is usually situated in a slope having shallow overburden where it tends to be unstable. The portal may be situated in such a position that has unsymmetrical earth pressure, slope failure, rockfall, debris flow, flood, avalanche, snowdrift, or fog. In such cases, additional minor adjustments of the portal position and installation of disaster prevention facilities shall be considered. The recommendable portal position is at the edge of a mountain ridge and perpendicular to the maximum angle of slope, in a stable slope with no danger of landslide.
3) When more than two tunnels are to be constructed parallel or crossing each other, the position of each tunnel should be determined, considering the effect the two tunnels will have on each other. The two tunnels will each other differently depending on rock condition or tunnelling method. The tunnels are affected each other when the distance between the two centres of the tunnels is more than twice the tunnel diameter (in hard rocks) or more than five times the tunnel diameter (in soft rocks). The distance of more than three times the tunnel diameter is applicable in ordinary conditions. If two tunnels are so close as to impact on each other or in soft rocks, the tunnelling method shall be reviewed and monitoring shall be planned.
The alignment of tunnels shall also be studied and determined carefully when the tunnel is proposed near other existing structures or when the surface ground is densely utilized with shallow overburden. The monitoring of ground surface settlement, subsidence, and vibration impact on structures and groundwater shall be planned. Protection and stabilization measures for the existing structure shall be planned.

(2) Vertical Alignment of Tunnels

A gradient of at least 0.2% is sufficient to allow natural drainage of water flow with good longitudinal drainage after completion of the tunnelling. During tunnel construction, a gradient of more than 0.3% is necessary for small inflow and up to 0.5% gradient for large inflow. Since the gradient affects the efficiency of mucking or material transportation, it is sometimes necessary to select the optimum value.

In road tunnels that need mechanical ventilation, keeping the steepest gradient less than 3% is desirable in order to reduce exhaust gas.

(3) Formation of Cross Section

Traffic envelope of roads tunnels is prescribed accordingly to type and classification. The tunnel shape shall be decided taking account of ventilation facilities, lighting facilities, emergency facilities, and road signs to the traffic envelope, and by allowing for tolerable construction errors. Inner cross-section of these tunnels shall be considered according to the cross-sectional gradient or cant determined by the horizontal alignment and traffic envelope. Examples of the formation of tunnel cross-sections are shown in Figure 1-1 and 1-2.

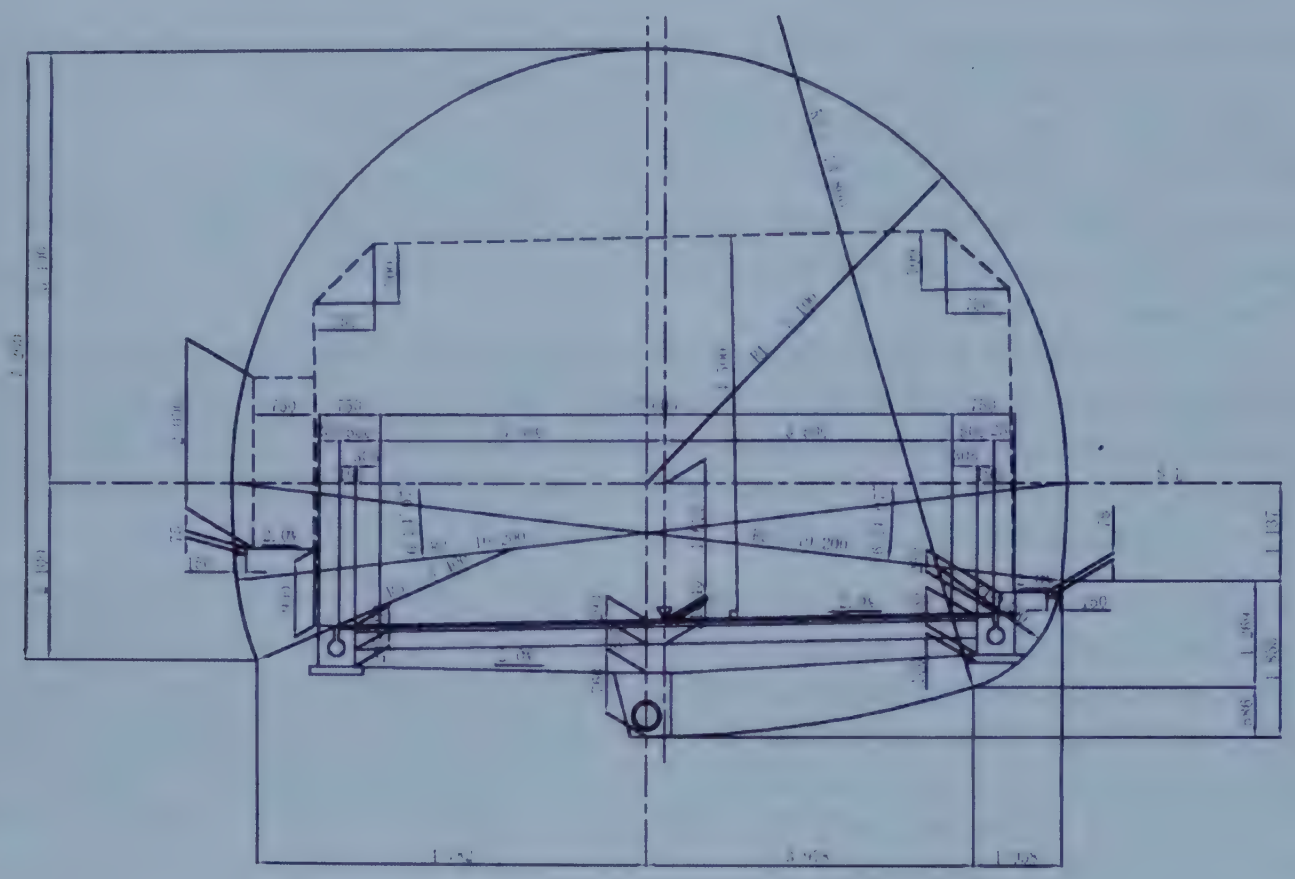


Figure 1-1: Example of Standard Cross Section of Road Tunnel (Two lanes with Inspection Path)

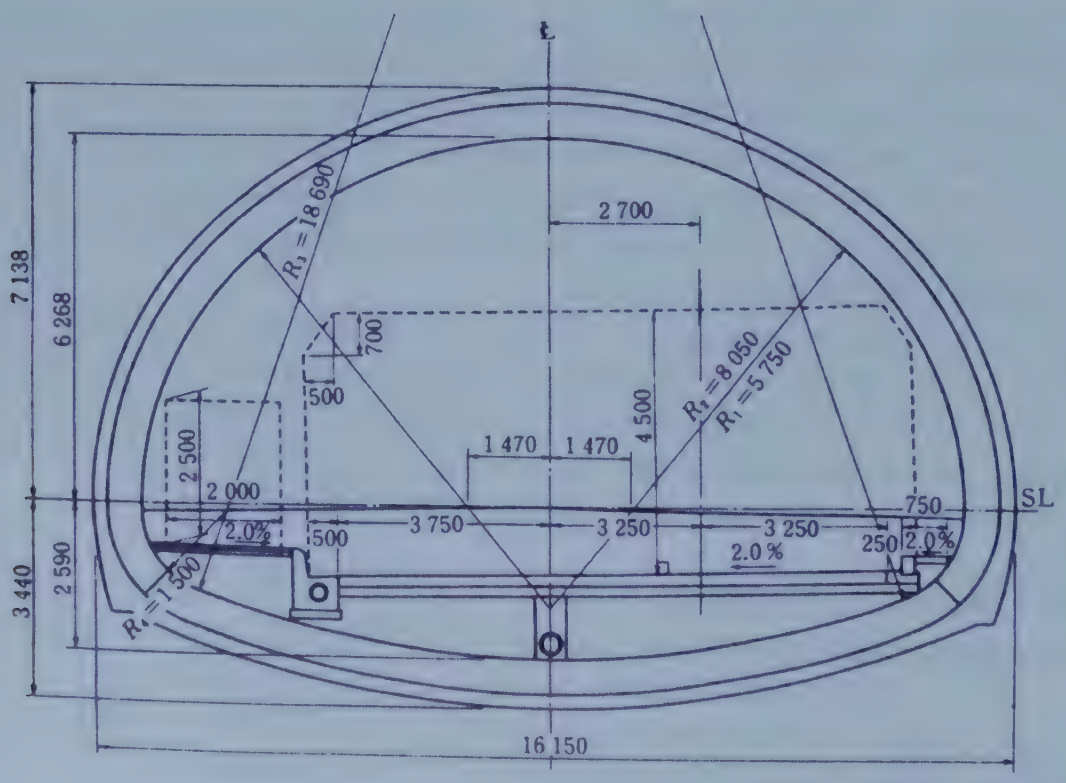


Figure 1-2: Example of Standard Cross Section of Road Tunnel (Three lanes with Sidewalk)

(4) Tunnel Facilities

In road tunnels, facilities such as ventilation, lighting, and emergency facilities are closely related to the amount of traffic, tunnel length, and longitudinal gradient. When designing the tunnel alignment and the position of portals, tentative design of ventilation points, position of ventilation ducts, ventilation scale, and emergency facilities shall be examined and compared comprehensively with construction cost, ease of construction, and maintenance cost. Especially, in the case of long tunnels, the arrangement of ventilation points or ventilation ducts may decide the tunnel alignment depending on the most economical ventilation system.

(5) Road Alignment and Structures outside of Tunnel Portals

It is necessary to consider traffic safety, economic efficiency, and driving comfortability with respect to the distance between tunnel portal/bridge and intersection.

<Road safety and economy>

It is necessary to alert the driver by installing "Warning signs/signboards" and "Guidance signs/signboards" that inform the driver in advance of the existence of an intersection ahead and the destination of the road. The signs/signboards should be placed approximately 150m before the intersection. The signs have to be possible to enlarge the size of the tunnel. Therefore, it will be desirable to keep the distance between the structure and the intersection as far as possible.

Figure 1-3: Warning and Guidance Signboards

<Driving comfortability>

It is necessary to secure a length/space for shifting lanes and stopping vehicles for safe and smooth guidance of vehicles before the intersection. The length required by these is due to the design speed of the road. When the design speed is V_s = 40 km/hr, a length of L = ℓ_t: 35m + ℓ_c: 15m + ℓ_s: 30m = 80 m will be required [1] . When this length competes with the structure, there will be a possibility that the size/section of the tunnel will be enlarged. It will be desirable to increase the clearance between the structure and the intersection.

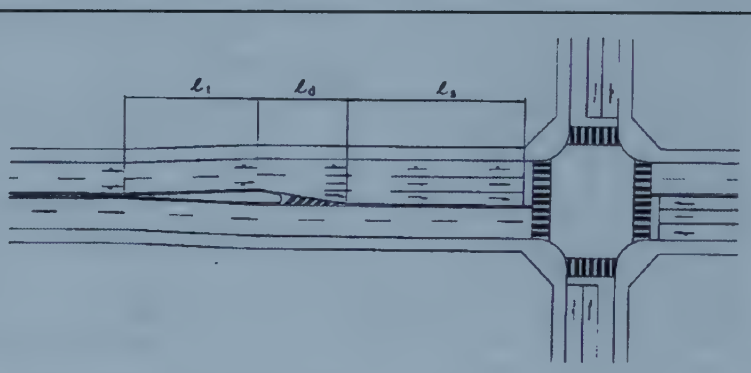


Figure 1-4: Length of Transition in Case of Lane

1.1.2 Work Plan

The work plan shall be prepared based on the purpose of the tunnel, its alignment, cross-section, rock conditions, location conditions, work safety, cost efficiency, and work period of the project.

(1) Work Section Division Method

When dividing the total length of a tunnel into work sections, each section should be provided with natural drainage, avoiding downward boring. Dividing into work lots and scheduling types of work should be effectively arranged so that the work sections can be finished at the same time as possible.

[1] The Planning and Design of At-Grade Intersections, Japan Society of Traffic Engineers, 1977

(2) Tunnelling Methods and Work Schedule

In general, the process of a tunnelling project is determined by putting together the periods of preparation works, tunnel excavation works, reinforcement works, and clearing. The total process should be able to include the times required to install the equipment. Figure 1-5 shows an example of a work schedule chart.

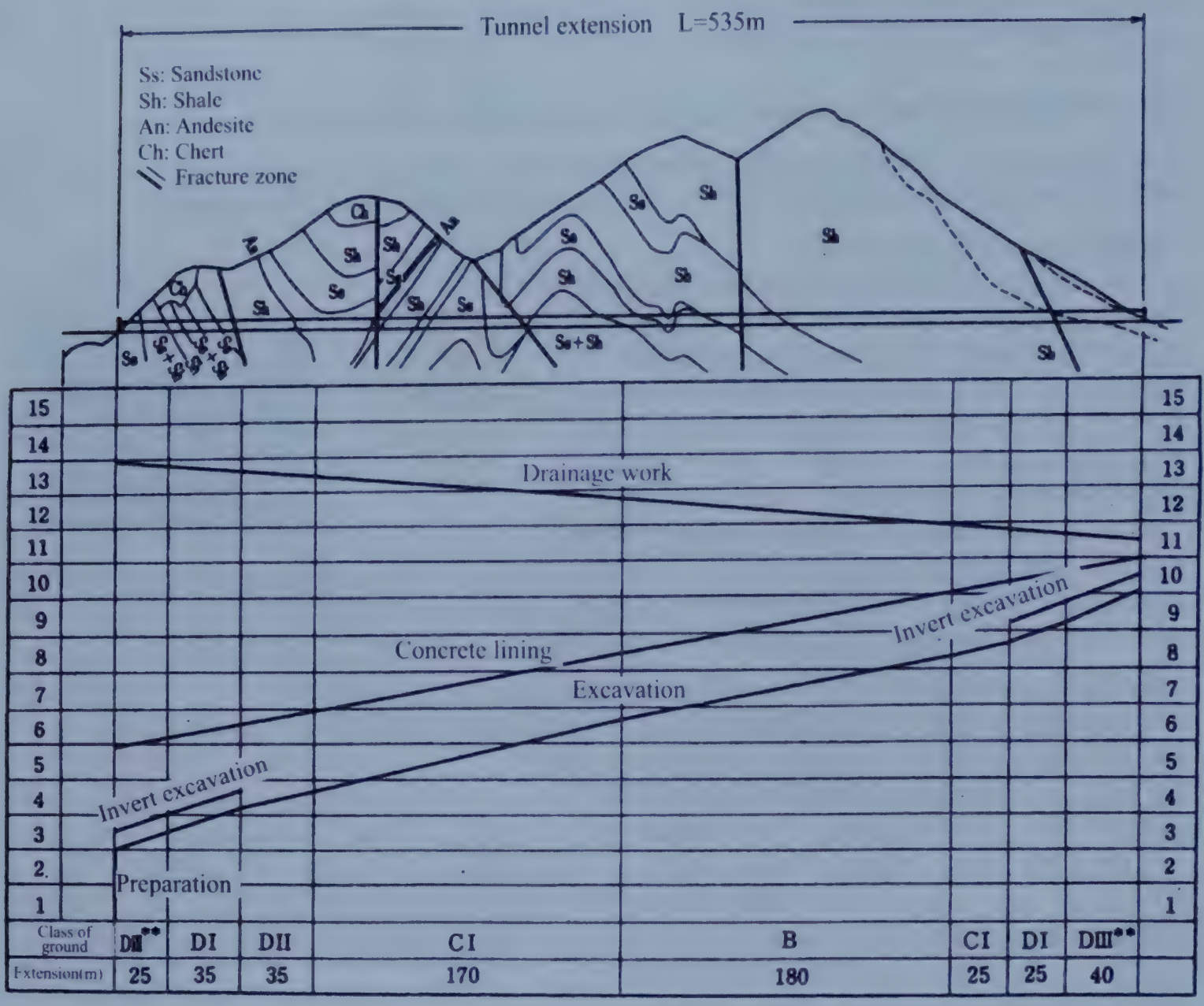


Figure 1-5: Example of Work Schedule Chart

(3) Shaft Tunnel

The shaft tunnel is used as a temporary facility for construction work only, or as a permanent facility for passage after the completion of a tunnel, or as a permanent ventilation shaft (facility annexed to the tunnel). When the shaft tunnel is used temporally for construction, consideration shall be given to various aspects, including reinforcement, refilling, and returning to the original state of the shaft tunnel portal site after the completion of the tunnelling project, and all possible measures should be taken to prevent adverse impacts on the main shaft and the surroundings. When the shaft tunnel is designed to be used as a permanent annex to the tunnel, its cross-sectional profile should be planned to meet the requirements for equipment design and for tunnelling work.

(4) Access Road and Muck Disposal Yards

If existing roads are used as hauling routes, sufficient discussion with the road manager, police, and roadside residents should be organized. Various supplementary solutions, if required, should be studied and implemented, including the construction of new

pavement, widening of roads, construction of branch routes, provision of crossing points, and school roads.

If there is no road or no available road in the proximity of a muck disposal area, construction of a new road is planned. For this plan, it is necessary to determine the horizontal and, longitudinal alignment of the road and the structure of the road, considering the purpose of the road, estimated traffic volume, types of vehicles to be used.

The main equipment to be provided outside the tunnel includes boring equipment (muck storage, track equipment, explosive-related equipment, and material storage), butcher plants, various buildings (office buildings, accommodations, warehouses, power plants, and repair shops), and electric equipment. Some cases need equipment to protect the environment such as noise-proofing equipment and wastewater treatment equipment. Table 1-1 shows examples of equipment provided outside the tunnel.

Table 1-1: Examples of Temporary Equipment provided Outside the Tunnel

Facility or Classification of work		Main Device	Notes
Drifting /Excavating Equipment	Power Equipment	Cooling pump	
	Repair Station	Welding Machine, Forging equipment, crane, jack.	
	Charge Equipment	Mercury is a rectifier or rectification monitor, crane, battery stand.	For battery car
	Muck Disposal area	Bulldozer, dump truck, lighting facilities	
	Loading facility	Derrick crane or goliath crane, wrecker is reserved.	
	Muck Disposal	Steel Hopper, Belt conveyor, feeder	
	Rail Facility	Storage track	
	Powder Facility	Embankment, powder magazine, powder work area, observation house, temporary road	Enough distance from the protected building
	Material storing site	Steel ribs, rock bolts, Wire net, etc.	
	(Access roads)		
Concrete manufacturing facility	Concrete mixing facility	Mixer, Weighing equipment	A hopper is necessary when using ready-mixed concrete.
	Cement silo	Screw conveyor	
	Aggregate silo	Belt Conveyor, clamshell	
Buildings	Office, lodging house, labour lodging house, warehouse, garage, laboratory, observation house, power equipment building, first-aid station, charging equipment building.	Water supply, lighting	Avoid slope failure, inundation, avalanche.
Electric power facility	Power incoming installation	Lead-in line of the high-voltage line, high voltage incoming panel, transformer.	
	Switchboard	High voltage power distributor, low voltage distributor, high voltage cable	
	Lighting facilities	Low voltage cable, lighting facilities of outside of tunnel.	
Environmental Preservation measures	Noise reduction	Soundproof wall, Silencer, sound isolation door	
	Water pollution prevention	Precipitation tank, thickener	
	Traffic safety measures	Widening roads, safe fence	

(5) Environment Preservation Measures

For environment preservation, there are several approaches, including the use of special tunnelling methods, pollution source-related measures, restriction of operation times,

protection, and strengthening of facilities (Table 1-2). It is necessary to select effective solutions when forming work plans. However, if the case does not allow the use of any of the above solutions, the possibility of alternative facilities should be studied, including the option of moving the facilities.

Table 1-2: Example of Environmental Preservation Measures

Impact on Surrounding environment	Examples of environmental preservation measures
Noise and vibration	- Source-related measures (use of low noise, low vibration type equipment, noise protection covers, silencing devices, installation of vibration protection devices, foundation strengthening, etc.) - Installation of sound-insulating devices (noise protection walls, doors, etc.) - Control of blasting time, adoption of controlled blasting and rock fracturing, etc. - Restriction of route and time of muck, material, and equipment transport
Damage to ground and structure	- Adoption of structure settlement prevention measures (inclined bolts, pipe roof, vertical pre-reinforcement, grouting, etc.) - Adoption of damage prevention measures for structures in vicinity (temporary underpinning, underpinning, earth anchors, pipe roof, grouting, etc.)
Groundwater loss	- Adoption of measures to prevent lowering of groundwater level (cut-off wall, pneumatic method, grouting, waterproof lining, etc.) - Provision of alternate water sources (provision of new wells or reservoirs, provision of piped water, etc.)
Contaminated water	- Provision of contaminated water treatment facilities (settling tank or precipitation of suspended matter by addition of coagulant together with dehydration treatment, PH regulation by addition of neutralization agent, oil absorption)
Traffic Obstruction	- Improvement of existing roads (Widening of roads, construction of new intersection and pedestrian passages) - New access roads.

1.2 Geotechnical Investigation

Rock conditions are examined at each stage of construction from planning to maintenance. During planning, the topographical and geological investigation for a selection of tunnel routes is executed. During the design and work plan, geology and hydrology are studied in detail. Since the characteristics of the earth, groundwater, and surrounding environment are closely related, the information gathered from studying topography, geology and hydrology shall be correlated with location conditions. All research shall be conducted in an early stage before construction and with a long-range view, and the results of each stage of investigation are to be fully utilized.
It may be difficult to obtain enough information on rock conditions before construction for long tunnels at great depths, so the geology shall be examined continuously during tunnelling. Figure 1-6 shows the flowchart of investigation of rock conditions and Table 1-3 shows the objectives of investigation.
Peculiar rock conditions, such as landslides, fault zones, etc., are shown in Table 1-4, or notable design and work conditions. Such as shallow overburden, porta areas as shown in Table 1-5 may have a great impact on tunnelling works. Therefore, in the early stage of route selection, the location and characteristics of the peculiar rock conditions and the detailed information of design and work conditions shall be fully examined and measure planned.

1.2.1 Survey for Feasibility Study

(1) Topographical Survey and Geotechnical investigation
The purpose of the investigation in the stage of tunnel route selection is to draw the overall rock conditions of the route and tunnel surroundings, with emphasis on searching for the special rock conditions as shown in Table 1-4. This will allow a summary of the topographic and geological problems in selecting the tunnel route. The results of investigation at this stage are important even in the subsequent investigation planning stage for the design and planning construction of the tunnel. The remaining problems to be settled later shall also be summarized at this stage.

(2) Geotechnical Investigation
The main purpose of the geotechnical investigation during tunnel planning is to grasp the overall geology of the area. The geotechnical investigation used for this research include documents search, aerial photograph analysis, surface geological survey, etc., as shown in Table 1-6
The items identified in investigation, objectives of investigation, usage of results, problems of investigation, and when to be applied are also shown in Table 1-6.

Table 1-3: Objectives and Grades of Investigation

Stage	Objective	Content of investigation	Range and grade of investigation
Investigation and Planning	- Selection of tunnel route	- Rock condition - Possible peculiar conditions - Rock condition	- Wide range including comparative route - Scale 1:25000 – 1:5000
	- Alignment - Location of portals - Preliminary design	- Topographical and geological investigation - Location and characteristics of peculiar ground	- Surrounding tunnel route and possible alternative site - Scale 1:5,000 – 1:1,000
Design	- Tunnel cross-section - Detailed design - Plan of construction	- Rock condition for design of supports - Information for work planning and cost estimation	- Tunnel route - Scale 1:1000 – 1:100
Construction	- Work control - Design change - Compensation	- Tunnel face mapping - Monitoring of tunnel deformation and tunnel support system - Environmental impact - Work record and record of rock conditions	- Inside tunnel and tunnel's sphere of influence
Operation and Maintenance	- Compensation - Maintenance and renewal	- Tunnel deformation - Surrounding environmental impact	- Inside tunnel and tunnel's sphere of influence

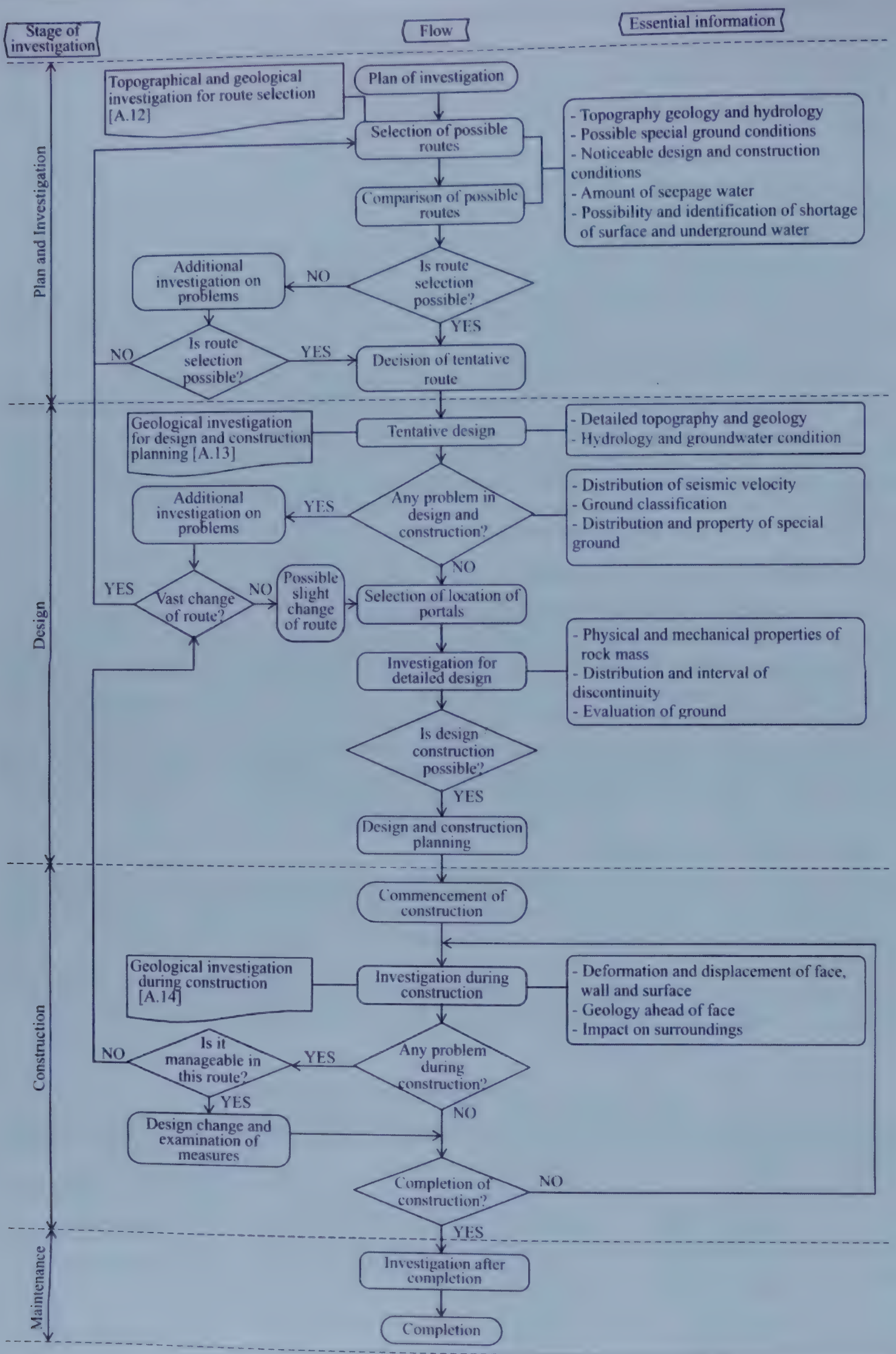


Figure 1-6: Flowchart of Investigation for Tunnelling

Table 1-4: Difficult Ground Condition for Tunnelling

Ground condition	Problem	Essential Information
Landslides/slope failures	Increase of earth pressure, Unsymmetrical pressure, Slope failure, Rock failure	Topography, Geology, Mechanical strength, Groundwater level
Fault fracture / folded disturbance	Face failure, Outbreak of groundwater, shortage of surface water	Mechanical strength, Seismic velocity, Competence factor, Groundwater level, Location of fracture zone
Soil / soft rocks	Tunnel face failure, Non-sustainable tunnel floor, Inflow of groundwater, Subsidence of floor, Subsidence of ground surface, Shortage of surface water	Mechanical strength, Relative density, Grain size distribution, Groundwater level, Water pressure, Hydraulic conductivity
Swelling rocks	Strong pressure on supports or lining, Decrease of tunnel shape due to squeezing of wall	Mechanical strength, Competence factor, Content of smectite, Natural water content, Degree of water immersion collapse
Rock Burst	Face failure	Mechanical strength, Brittleness, Acoustic emission
Geothermal heat, hot spring, toxic gas, heavy metal, etc.	Eruption of high pressure and high-temperature water, or toxic gas, leaching of heavy metal or acidity water	Temperature, Concentration of gas or oxygen, Content of heavy metal, pH
High water pressure or large inflow	Outbreak of groundwater, Increase of earth pressure, Unsymmetrical pressure, Face failure, Shortage of surface water, Water, pollution	Groundwater level, Amount, and pressure of groundwater

Table 1-5: Notable Design and Tunnelling Conditions

Design and tunnelling condition	Problem	Essential information
Shallow overburden	Subsurface of surface, Cave-in, Unsymmetrical pressure	Topography including landslide, Mechanical strength, Hydraulic conductivity, Location condition
Urban area	Subsidence of surface, Displacement, and deformation of adjacent structures, lowering of groundwater level	Mechanical strength, hydraulic conductivity, Location condition, Location of adjacent structures
Large depth	High-pressure groundwater, Increase of earth pressure	Mechanical strength, Hydraulic conductivity, Pressure, and level of groundwater
Underwater	Large quantity of groundwater	Mechanical strength, Amount of groundwater, Hydraulic conductivity, Topography of bottom
Inclined drift or shaft	Groundwater, Inflow of surface water and soil from the portal	Mechanical strength, Amount of groundwater, Hydraulic conductivity
Portal	Landslide, Slope failure, Unsymmetrical pressure, subsidence of cave-in of surface	Mechanical strength, Topography including landslide
Construction in the vicinity	Unsymmetrical pressure, Deformation of adjacent structures	Mechanical strength
Large cross-section	Face failure	Mechanical strength

Table 1-6a: Geotechnical Investigation

Method	Items identified for investigation	Usage of results for planning and design	Problems of investigation	Stage: Selection of route	Stage: Design and construction planning	Stage: Construction
Document search	1. Geography, geology and hydrology of planned area. 2. History of past disasters and outline for possibility of construction.	1. Problems or problematic areas are made clear at route selection stage. 2. To extract problems that need to be made clear at surface survey or following precise investigation.	1. There may be no data available depending on site. 2. Generally maps are not precise enough. 3. Information on maps is not always related to purpose of examination.			
Aerial photograph analysis	1. Unstable ground around portal area. 2. Structure of geology, especially lineament such as a fault. 3. Surface geology before any artificial change.		It provides standard precision but double- checking by surface survey is necessary.	◎	○	△
Surface geological survey	1. Distribution, characteristics, and stability of surface geology. 2. Distribution and characteristics of foundation ground. 3. Distribution and shape of geological structure.	1. To make horizontal and cross sectional geological map and to clarify distribution and characteristics of ground surrounding route. 2. Qualitative evaluation of safety and construction possibility of route's peripheral geology.	1. A geological map is only one of interpretation of results of investigation; it has to be verified in later investigation. 2. It is greatly affected by precision of geographical map used.			
Geophysical prospecting: Seismic prospecting	1. Elastic wave velocity of ground. 2. Position, scale and velocity of slow velocity zone.	1. To grasp thickness and characteristics of unconsolidated sediments, weathered layer. 2. To grasp condition of rock mass at excavation level. 3. To grasp position, scale, condition and continuity of fault, fractured zone, and soft layer. 4. Examination of ground classification.	1. When slow velocity area is narrowly included it reduces precision of analysis. 2. Evaluation of geological distribution, level of weathering, level of fracture development has to be combined with other investigations and evaluated synthetically.			
Geophysical prospecting: Electrical prospecting	- Specific resistance of ground and cross sectional distribution of specific resistance.	1. To grasp depth of talus and weathered layer. 2. To grasp geological structure and characteristics from specific resistance. 3. Detection of narrow band of soft layer. 4. To grasp properties and scale of fault. 5. Examination of distribution and properties of groundwater and aquifer.	1. Geographical and geological properties of site significantly influences precision of investigation. 2. It is not directly related to mechanical strength of ground.	○	○	△
Boring Investigation	1. Stratification and distribution of sand and rock. 2. Position, scale, properties, continuity of fault, fractured zone, and soft layer. 3. Type of rock, weathering and alteration, properties of fracture and joint. 4. Possibility of groundwater, pressure and amount of water inflow.	1. Geology is directly investigated to clarify details of distribution and properties of ground. 2. To examine ground classification, excavation methods, support and lining, and amount of explosives.	- It is a pinpoint investigation and needs to be combined with a surface survey, geophysical prospecting, etc.			◎

Table 1-6b: Geotechnical Investigation

Borehole test or logging	Standard penetration test	1. Examination of stability around portal and places where overburden is small. 2. To grasp depth of rock and bearing layer.	1. N-value, softness and / or tightness. 2. Sampling and categorization of soil.	1. It is generally not applicable to rock and gravel layer. 2. When N-value is greater than 50, a fine distinction is not possible.	○	○	△
	Borehole load test	- Deformation coefficient, elastic coefficient, etc.	- Analysis of ground deformation.	1. Selection of test equipment appropriate for geological conditions and borehole diameter is necessary. 2. Selection of section that includes typical geological situation by core observation is necessary.	○	○	△
	Permeability test	- Hydraulic properties of ground	1. Evaluation and forecast of sudden and / or steady water inflow at aquifer. 2. Evaluation of face stability in unconsolidated ground.	1. Measured values are rough estimates that need to be interpreted by value of exponent. 2. Selection of test appropriate for ground condition is necessary.	○	○	△
	Velocity logging PS logging	- Vertical distribution of elastic velocity of ground.	1. Indirect estimation of rock / sand classification, or level of weathering, alteration, and fracture development. 2. To grasp low velocity layer.	1. Sometimes it is not measurable when there is no groundwater. 2. Sometimes low velocity layer is not measurable below groundwater table.	○	○	△
	Electrical logging	- Apparent specific resistance of borehole walls.	1. To grasp geological distribution by specific resistance. 2. Evaluation of groundwater condition in aquifer.	1. It is limited to below groundwater table. 2. It is not measurable in casing penetrated section	○	○	△
	Density logging	- Density of ground.	- Value and distribution of wet and dry density, and void ratio of ground.	1. Relatively strong γ-ray is necessary. 2. Examination experiment is in need to make calibration curve.	○	○	△
	Caliper logging	- Change of diameter of borehole.	1. Qualitative estimation of ground stability. 2. Vertical distribution of unstable stratum.	- Measurement can't be done in borehole with casing.	○	○	△
	Groundwater prospecting in borehole	- Groundwater fluid stratum.	- Distribution of groundwater fluid stratum.	1. Single electrode method takes long time to measure. 2. In the case of deep logging, it is hard to stir the dissociation material. 3. Measurement can't be done in borehole with casing.	○	○	△
	Borehole TV	1. Condition of stratification Scale, properties, and strike dip of fault and fractures. 2. Condition of water inflow	1. Examination of face stability by condition of fractures and their strike / dip. 2. Examination of condition at water inflow area.	- Complete and thorough cleaning of borehole is necessary.	◎	◎	○
	In situ stress measurement	- In situ stress of ground.	- Stress condition of ground.	- Only partial, not global, stress condition can be obtained.	△	△	△
	Laboratory test	1. Physical and mechanical characteristics of constituting rock: unit weight, elastic wave velocity, compressive strength, etc. 2. Mineral characteristics of constituting rock: clay mineral content, slaking characteristics, etc. 3. Physical and mechanical characteristics of constituting soil: particle size distribution, water content, compressive strength, consistency, etc.	1. To grasp level of deterioration of rock by fractures etc. 2. To grasp mechanical characteristics of rock. 3. Evaluation and forecast of squeezing ground. 4. Estimation of stability of unconsolidated sediments. 5. Examination of face stand-up stability at unconsolidated ground.	1. Since sample does not have fractures value is not a good representation of ground. 2. Experimental result varies greatly due to water content of soft rock samples. 3. Mechanical experiment tends to be significantly affected by disturbance of sample during sampling.	○	◎	○

1.2.2 Geotechnical Investigation for Design and Work Plan

(1) Purpose of Geotechnical Investigation

The geotechnical investigation is to clarify the following items;

a. The overall geology, distribution, and characteristics of rocks along the tunnel.

b. Rock classification with technical consideration based on the results of the investigation.

c. Topography and geology of portal locations, basic references to the problems and measures.

d. Evaluation of face stability, design of support selection of reinforcements, basic references to the selection of excavation method and tunnel driving method.

e. Distribution and characteristics of peculiar rocks, predictions of potential problems and phenomena, and basic reference to possible reinforcement works.

(2) Investigation Items

Investigation items and methods to be done in the stage of design and work plan are listed in

. Investigation items and methods are chosen according to the rock condition by referring to Table 1-7. Characteristics and problems of general geological/geotechnical investigation for tunnels are shown in Table 1-6. Characteristics, identified items, usage of results, and problems obtained by the investigation shall be recognized and executed carefully in multiple combinations.

(3) Geotechnical Investigation on Peculiar Rock Conditions

Significant points to investigate for peculiar rock conditions and notable design and construction conditions are shown below. Items to be investigated for peculiar rock conditions are listed in Table 1-7.

1) Geotechnical Investigation in Peculiar Rocks

a. Tunnelling in landslide

Portals or tunnels having shallow overburden such as in dells or near slopes shall be investigated for the possibility of landslide or slope failures, and the stability of the tunnel. Investigation is required in order to design slope stability works. When the potential for landslides is great, the relocation of a tunnel alignment to avoid the landslide is studied.

b. Tunnelling in fracture zone

Shear zone of fault can be detected by geotechnical and geophysical investigation during route selection. A fault fracture zone may contain groundwater. The determination of the characteristic and distribution of fracture zone is important. The characteristics of a fracture zone can be estimated with the results of seismic surveys, geotechnical borings, etc..

Active fold where deformation is in progress in the Tertiary Green Tuff area is most difficult in Japan. If large tectonic forces are acting on the axis of the fold in addition to the overburden pressure, investigation of swelling rocks, described below, may be necessary. The tectonic forces may be also acting in the Himalaya region.

c. Tunnelling in soil

Loose or weakly cemented sandy soil and gravelly soil that constitutes Diluvium or part of Alluvium layers, and unconsolidated volcanic material such as volcanic ash or gravel and pumice stone are classified into soil. Shallow overburden in groundwater-rich loose soil may cause tunnel face collapse, a large amount of water

inflow, surface settlement, or cave-in. Loose sand or gravel layer is simple in its structure, therefore surface geological survey, geotechnical borings, borehole logging, and groundwater survey can estimate the geological characteristics and groundwater inflow. Especially, in the case of homogeneous fine sand, the soil laboratory test for the index of liquefaction is recommended. In the case of an aquifer interspersed with impermeable clay, the groundwater in the aquifer can be pressurized.

d. Tunnelling in swelling rocks

Swelling rocks are divided into two categories, i.e. general squeezing and swelling. The former phenomenon is caused by plastic flow due to large stress, and the latter is caused by rock expansion. Rock swelling is caused depending on the rock type or rock characteristics, and stress in rocks by geological structure and static earth pressure. Rock swelling can be seen generally in tertiary mudstone, tuff, fault gouges, fracture zone, solfataric clay, serpentinite, etc.

Evaluation of squeezing index for the tertiary mudstone or tuff is determined by the laboratory tests shown in

. Generally, the strength of the rock is so low, that in-situ tests using boreholes are more effective.

e. Rock burst

A rockburst is a spontaneous, violent failure of rock that can occur in high-stress deep tunnels. Rock bursts result from brittle fracturing of rock, causing it to collapse rapidly with violent spalling of rock. The opening of a tunnel relieves surrounding rocks of tremendous pressure, which can literally cause the rock to explode as it attempts to re-establish stress and strain equilibrium. The released energy leads to a violent fracture of the surrounding rock around the excavation and reduces the potential energy of the rock, which makes the surrounding rock reach another equilibrium state. AE measurement is sometimes used to predict rock bursts during construction.

f. Tunnelling in geothermal heat, hot springs, toxic gases, heavy metals, etc

Sometimes, groundwater is isolated in fault gouges, large uncemented soil, joints or cracks of rock mass, and friable limestone and volcanic rocks. Geotechnical boring and hydrogeological investigation shall be employed to find fault fracture zones and uncemented soil and hydrological conditions. Geophysical surveys such as seismic or resistivity surveys should also be employed depending on the geology.

2) Geotechnical investigation in particular conditions

a. Shallow overburden

In the ground with shallow overburden, forming an arch is difficult, surface subsidence or cave-ins occur easily, and preservation of face and wall is sometimes frustrating. Especially, if there are some structures on the ground surface, data to estimate the value and range of settlement shall be obtained. If surface settlement is to be restrained, the strength and deformation characteristics of the ground are necessary for the prediction of surface settlement. Standard penetration test, in-situ test using borehole or laboratory tests are to be conducted.

b. Tunnelling in urban area

Tunnelling in urban areas is severely restricted by surrounding conditions and geological conditions compared to the usual mountain tunnelling. In this case, conventional tunnelling methods such as open cut or shield tunnelling method is popular in urban area, but recently the mountain tunnelling method is increasing. It is important to investigate the surrounding conditions to determine if there are surface and underground structures that will hinder tunnelling. Since the overburden of tunnel may be shallow and loose/soft material in urban tunnelling, suitable investigation shall be selected and conducted.

c. *Inclined and vertical shafts*
Investigation for inclined shafts is almost the same as that for main tunnels. The investigation for groundwater must be executed in order to prevent flood in the shaft and to plan of drainage of inflow water. The investigation method for vertical shaft is geotechnical boring and borehole logging, including in-situ test for groundwater.

d. *Portals*
One of the most difficult tasks in tunnelling is the construction of portals. Not every portal condition is the same, so each portal must be studied individually. In case of portals are in landslide or slope failure risk, the corresponding investigation which is usually vertical or horizontal boring shall be conducted based on the risk anticipated.

e. *Construction in the vicinity*
When a tunnel is constructed near existing structures, the location and characteristics of existing structures, environmental, geological conditions of the surrounding area protect the existing structures from tunnelling works. In the case of twin tunnels, the ground behaviour due to mutual interference of tunnelling is important. The surrounding ground displacement, vibration due to construction, and change of the groundwater table shall be predicted.

f. *Large tunnelling sections*
In recent years, large tunnelling sections, e.g. over 180m^2, have been constructed. These tunnels are researched taking rock conditions and surrounding conditions into consideration as in normal tunnelling sections. Reinforcements tend to be employed in large-diameter tunnels, therefore investigation for reinforcement is required. Geological probing ahead of the face from inside the tunnel is recommendable.

Table 1-7: Ground Condition, Investigation Items and Investigation Methods

		Ground Condition	Topography			Geology		Rock or Soil Quality					Groundwater			Physical Property		Mechanical Property		Mineralogical & Chemical property			Others			
		Investigation items / Investigation Methods	Landslide Collapse	Unsymmetrical pressure	Overburden	Geological mapping	Fault or fold	Type of rock / Soil	Lithology	Discontinuity	Weathering or alteration	Degree of consolidation	Aquifer	Groundwater level	Hydrological conductivity	Seismic velocity	Physical property	Compressive strength	Deformation coefficient	Clay mineral	Slaking property	Water absorption and swelling rate	Geothermy	Hot spring	Toxic gas	Heavy Metal
General ground		Hard rock or medium hard rock	++	++		++	++	++	++	++	++		++	++		++	++	++					+	+		+
		Soft Rock	++	++	++	++	++	++	++	++	++		++	++	++	++	++	++	++	++	++	++	+	+	+	+
		Soil	++	++	++	++	+	++				++	++	++	++		++	++	++							
Special Ground		Landslide or possible slope failure at tunnel portal	++	++	++	++		++			++	++		++		++	++	++		++						
		Shallow overburden			++	++		++			++	++	++	++	++		++	++	++†							
		Fault failure zone or folded disturbance zone	++		++	++	++	++	++	++	++	++	++	++	++	++	++	++	++	++	++	++		+	+	
		Unconsolidated ground			++	++		++				++	++	++	++		++	++	++							
		Swelling rock	++	++	++	++	++	++	++	++	++	++				++	++	++	++	++	++	++			+	
		Rock burst			++			++	++	++						++	++	++								
		Geothermal heat, hot spring, toxic gas. heavy metal, etc.			+	++	++	++	++	++	+		++	++		+	++	+	+	+		+	++	++	+	+
		High pressure or large inflow				++	++	++	++	++	+	++	++	++	++	++										
Stage of route selection		Document research	++	+	++	+	+	+	+														+	+	+	+
		Aerial Photograph search	++	+		+	++		+	+																
		Surface geological survey	++	++	++	++	++	++	++	+	++	++	++										+	+	+	+
		Seismic survey	+			+	++			++	++	+				++									+	
		Resistivity survey	+			+			+		+	+	++	++			†						+	+		
		Geotechnical boring	++			++	++	++	++	++	++	++	++	++									+	+	+	+
Stage of design and construction planning – Construction	Borehole test and Logging	Standard penetration test										++						+	+							
		Lateral load test																	++							
		Permeability test											++		++									++		
		Seismic Logging				+	+			++	++	+				++	+									
		Resistivity logging				+		+			+	+	++	+	+		+									
		Borehole TV					+			++																
	Laboratory test	Unit weight test															++									
		Natural water content test															++				+	+				
		Gain size distribution test															++		++							
		Test of specific gravity of soil particle															++		+							
		Consistency test															++		+			+				
		Unconfined compression test								+	+							++	+			+				
		Triaxial compression test																++	++			+				
		Brazilian test																+								
		Point load test																++								
		Needle penetration test																++								
		Permeability test													++											
		Ultrasonic wave test														++	++	+								
		Slaking test																		+	++	+				
		CEC test																		++	+	+				
		Swelling test																		+	+	++				
		X-ray Diffraction test																		++						++
Additional Investigation						++		++	++	++	++	++	++	++		+	+	+	+			+	++	++	++	

1.2.3 Geotechnical Investigation during Tunnelling Works

In the stage of tunnel route selection or design and construction planning, often the geology cannot be fully researched depending on the ground surface condition of other conditions. Sometimes, unforeseen geology is encountered or unexpected problems arise. In this case, necessary geological investigation shall be conducted as soon as possible as occasion demands even during the tunnel construction stage. Geotechnical investigation during construction is listed in Table1-6. Appropriate geotechnical investigation is to be selected depending on the situation of geology and tunnelling method.

1) Geotechnical boring
Geotechnical boring or logging from ground surface shall be added if geological conditions, detailed distribution of aquifer, and water pressure is acknowledged during tunnelling.

2) Seismic survey
The seismic velocity of the tunnel formation-level sometimes cannot be obtained precisely by the seismic refraction survey in the stage of plan and design. The difference between predicted and real geological conditions is so large that the accuracy of seismic velocity can be improved by measuring the seismic velocity between tunnel wall and ground surface or borehole.

3) Probing ahead of face
Probing ahead of the face is the primary method to obtain the geological information, in cases where there is concern about geological conditions ahead of the face. Especially, this measure is most effective when the face tends to collapse due to fracture zone or aquifers with high water pressure. Major probing methods are as follows;

i) Surface geotechnical survey
Based on the geotechnical information from the observation of the face, the geology ahead of the face can be detected by another surface geological survey.

ii) Geological observation of face
Geological mapping in tunnel longitudinal and plane figures by the geological observation of the face may identify the geological change ahead of the face. Strike and dip of bedding, fault fracture zones, alternate zones shall be measured to correct the geology ahead of the face in preliminary design.

iii) Advancing boring
Advancing horizontal boring from the tunnel face can directly identify the geology or seepage water condition ahead of the face and can be used as ad drainage borehole.

iv) Geophysical survey
Geophysical survey for probing ahead of the face is classified into two types, i.e. seismic survey and resistivity survey. Seismic reflection is used to catch the reflected wave from the plane between rocks with different seismic wave velocities, prospecting the position of geological discontinuity or fault fracture zone. Underground radar using electromagnetic wave can also be used for predictions.

1.2.4 Hydrological Research

Inflow water in the tunnel affects the tunnelling works and sometimes causes a water shortage in the vicinity area. Hydrological research shall be beneficial to predict the type and volume of inflow water, examination of problems concerning design and tunnel construction due to inflow water, evaluation of the impact on surrounding environment,

and examination of the problems concerning operation and maintenance of the tunnel after completion.

1) Prediction of inflow water in tunnel
Inflow water in tunnel is classified into two categories; concentrated inflow at the stage of construction and steady inflow after completion. The type and volume of seepage water and its catchment area depend mainly on the structure of the aquifer that contains groundwater and the characteristics of the aquifer. The prediction methods for the amount of seepage water and catchment area are as follows.

i) Case-study of tunnelling on the surrounding area or in similar geology

ii) Method using topographical or hydrogeological conditions

iii) Method using hydraulic formula

iv) Numerical analysis by hydrogeological model

2) Problems of design and construction
Seepage water in tunnels is not only an obstacle to excavation, but is also a critical item affecting the safety and cost of construction, and may cause the face to collapse. Planning of the excavation method or water treatment facilities and measures for safe construction shall be examined predicting the place or amount of concentrated inflow in the stage of route selection or design and construction plan because the problems for design and construction plan depend on the concentrated seepage water at the beginning of excavation. Measuring the amount of inflow and comparing it with the original predicted value are necessary to advance the excavation. Observation shall be continued even in the stage of operation and maintenance after the completion of tunnelling.

3) Impact on the surrounding
The impact of water inflow in a tunnel on surrounding surface water or groundwater is a serious problem in the community with regard to environmental preservation. It is mainly due to the steady seepage of water during construction. During route selection or design and work plan, measures against drying up of surface water or groundwater shall be studied beforehand, predicting and evaluating the extent of the impact and modifying the tunnel route in the worst case. In the stage of construction and operation/maintenance after completion, the survey of water balance including inflow water shall be continued. The impact of tunnelling works on the surrounding environment shall be continued, and the measures to prevent the impact from expanding shall be examined and implemented.

4) Step-by-step implementation of hydrogeological investigation
Hydrological investigation is categorized into hydrogeological investigation, water inflow-outflow balance investigation, and hydrological environmental investigation. Results of the investigations and additional results from document research or case-study research shall be examined and various predictions and evaluations shall be made.
Table 1-8 shows the investigation purpose, content, and appropriate stage at which it should be implemented.
Hydrological conditions shall be investigated at every stage of tunnelling depending on the occasion in a series of investigations from the stage of route selection to the stage of operation and maintenance after completion. Accuracy shall be improved by repeating the predictions, evaluations, and modifications. The hydrological investigation during the stage of route selection to the stage of

design and work plan, is very important because sometimes there are no fundamental solutions for water problems after the determination of tunnel route.

Table 1-8: Overview of Hydrological Investigation

Item		Purpose of Investigation	Content	Investigation Stage			
				Route Selection	Planning	During Construction	After Completion
Categories Hydrological investigation	Data	To plan research by examining hydrogeological structures, an overview of groundwater, and anticipated problems	Geography and geology: Hydrogeological structures hydrological metrology: rainfall, temperature, Etc. Usage of groundwater: Wells, irrigation, etc.	++	++	-	-
	Case Study	Evaluation of water inflow and extent of decrease/shortage of water at target tunnel. Examination of the suitability of various research methods.	Data from the previous tunnelling projects Geology, water inflow amount, Work conditions, an area that was affected by the shortage of water, Preservation measures	++	++	-	++
	Hydrogeological Investigation	<Structure of Aquifer> Hydrogeological structure, properties of groundwater, etc. are summarized in a hydrogeological chart to forecast the spot of water inflow and water intake area.	Surface geological survey	++	++	+	-
			Geophysical survey	+	++	+	×
			Investigation boring	+	++	+	-
			Borehole logging	-	++	+	-
			Water quality tests (In-Situ, Laboratory)	-	+	+	+
		<Characteristics of aquifer> Hydraulic coefficients such as permeability, storage coefficient, etc. are evaluated by hydraulic theories to forecast water inflow amount and water intake area.	Single borehole permeability test (such as piezometer method)	-	++	-	-
			Water inflow pressure test, injection test	-	++	-	-
			Pumping test, Injection test	-	+	-	-
			Tracer test, flow direction, and velocity test	-	-	-	-
			Water depth reduction test	-	-	-	-
	Water Balance	Hydrological meteorology, surface water flow amount, groundwater table, etc. are reached to examine water balance, in order to forecast groundwater behaviour during construction.	Hydrological meteorology: rainfall, temperature.	++	++	++	++
			Surface water flow: the amount of river flow, lakes, and reservoirs, irrigation, springs, etc.	++	++	++	++
			Groundwater table: observation wells, existing wells.	++	++	++	++
			Evaporation rate	+	+	+	+
			Inflow in tunnel, impact on shortage of water	-	-	++	++
	Hydrological environment	To examine water resource and water usage in areas of water intake and vicinity areas, in order to forecast the impact of the tunnel.	Water resources: Spring, river, lakes, reservoirs, wells, effective rainfall	++	++	+	+
			Water usage: Water supply and sewers, industrial and agriculture water	+	++	+	+
Method of forecast		To forecast the possibility of water inflow, its amount, location, and water intake area.	Past construction example	++	+	-	-
			Geology and hydrogeological conditions	+	++	-	-
			Hydraulic formulas	+	++	-	-
			Numerical analysis	-	-	-	-

++ : Required + : Recommended - : If necessary

1.2.5 Geotechnical Reports

Investigation results shall be described, clearly distinguishing facts and interpretations, and unsolved problems shall be arranged so that they may be evaluated for importance. In many cases, ground and location conditions are investigated by using the means which are suitable for their respective purposes. Since the results have a certain relationship to each other, they should be examined and evaluated from a comprehensive standpoint, with consideration for accuracy of the results. Since the results of the hydrological investigation are related to the results of ground and location conditions, they need to be examined and evaluated in the process of arrangement, considering the overall accuracy of each investigation before being integrated, as established data, into the investigation results of ground and location conditions.

The investigation results of the rock conditions should be organized in an appropriate manner according to the stages and purposes shown in Table 1-9, and arranged to

facilitate the use of data. In particular, any changes in the cross-section of a tunnel, excavation method, reinforcement methods that have already been determined in the original design, may have a significant adverse impact on the work period and construction cost, as well as on safety. To eliminate this inconvenience, it is important that the investigation results of the rock conditions should be effectively arranged from an engineering viewpoint, so that they may serve as materials for assessing specific work plans, designs, and construction.

In process of organizing investigation results, it is imperative to comprehensively evaluate various kinds of results that are obtained from past investigations and tests, including already available documents. Investigation results should be put together into various categories such as rock conditions of the whole tunnel so that they can be utilized effectively as reference documents for alignment selection and design/construction plans of the tunnel. The geological distribution and characteristics, frequently problematic, should be established in the form of geological maps, and geological longitudinal profiles.

Table 1-9: Items of Investigation at Each Stage

Investigation stage	Item
Planning for alignment	Identify obstacles that may cause problems for the construction project, examination material of planning for alignment and method selection.
Design and construction planning	Various conditions such as topography and geology, ground classification, special condition, reinforcement works
Construction	Ground stabilization of the face, quantity of water inflow, displacement magnitude, predictions, and results such as the rock condition, examination of excavation methods and reinforcement works, etc.
Maintenance	Earth pressure, deterioration of rocks and materials, water leakage, deformation by forest damage, ground subsidence, impact on the groundwater, noise, and vibration after public use, etc.

The results of the investigation of rock conditions shall be organized into the following sections so as to be easily utilized in studies concerning design and tunnelling methods.

g. Objective, location, scope, and method of the investigation and name of the person in charge
h. Summary description of topography and geology, together with geological structure
i. Geological maps
j. Geological cross-sections
k. Geological profile

The geological profile shall record the results of the investigation including the type of rock, its properties, seismic wave velocity, competence factor, and state of water inflow (See Figure 1-7).

Special attention that might pose problems shall be individually itemized and classified according to the possibility of occurrence and importance, including solutions. Tunnelling sections needing a geotechnical model-based rock evaluation should be identified.

Typically, the geological profile shown in Figure 1-7 is prepared for most of the cases. This geological profile clarifies geological structure, as well as seismic wave velocity, qu/P_v ratio, state of groundwater. In general, the ground is classified according to classes of rocks.

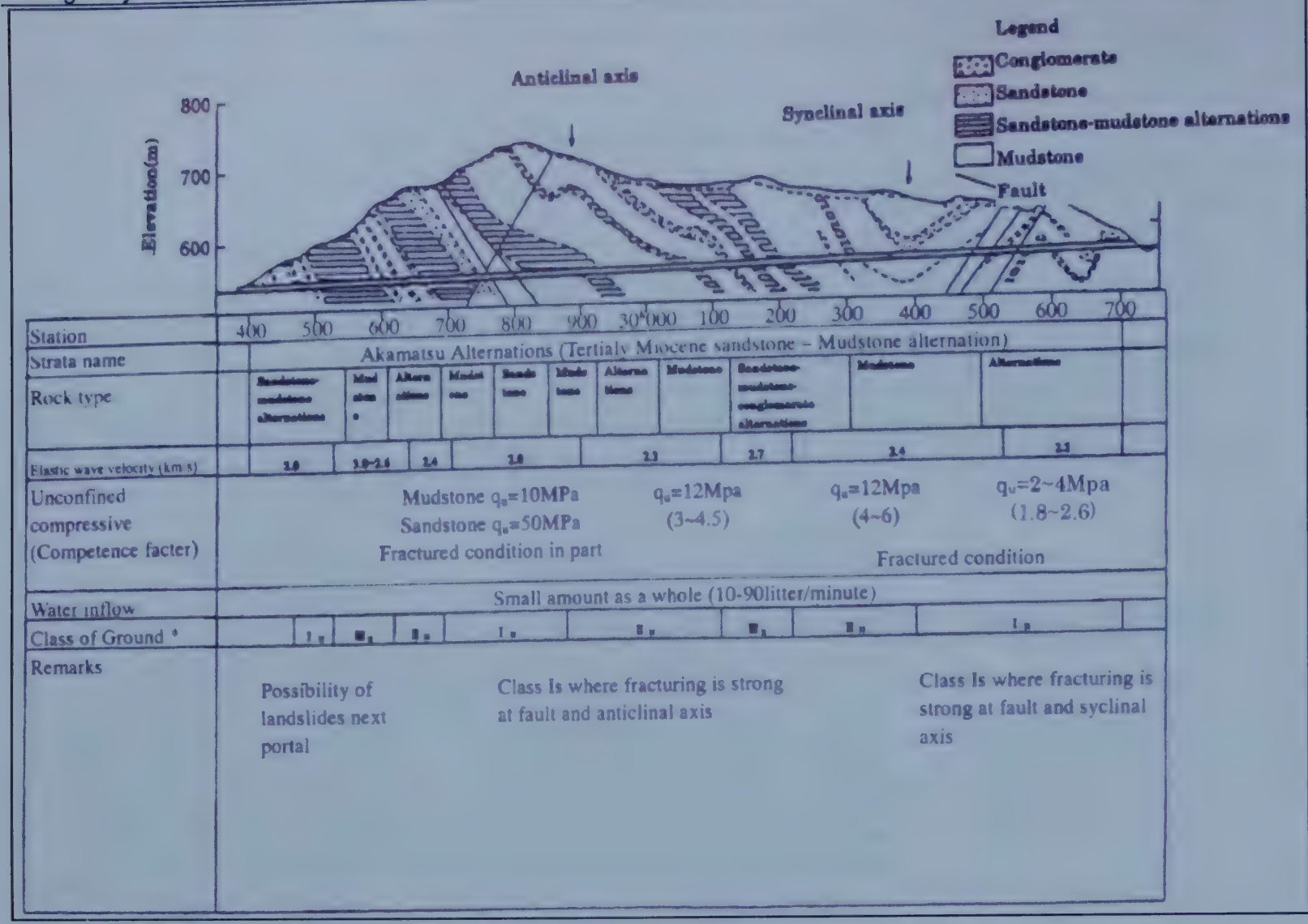


Figure 1-7: Example of the Longitudinal Geological Profile

1.2.6 Investigation for Environmental Preservation Measures

It is necessary to minimize adverse impacts of tunnelling on the natural environment (ground surface water, groundwater, flora, and fauna), social environment (land utilization, cultural assets, etc.), and human living environment (noise, vibration, and air pollution). The items to be investigated for preserving the environment should include relevant items from the planning stage (to those in the maintenance stage), and cover the areas surrounding the tunnel route.
The planning engineer in charge must as soon as possible begin researching the laws and regulations relating to the construction of the tunnel concerned, and regarding responsible compensation coverage.

1) Environmental investigation
The main environmental problems by tunnelling are noise, vibration, drying up of wells, turbid water, and damage to the buildings. The environmental impact prior to and during the tunnelling works shall be predicted and evaluated, and p appropriate measures shall be studied. The potential environmental problems that may arise during the construction and subsequent use shall be studied in order to understand how the environment was affected by the tunnelling works.

a. Table 1-10 shows environmental investigation items and investigation areas.
b. When the investigation results indicate that there is a fear of noise, vibration, draught, ground subsidence, muddy water, air pollution, and traffic problems, for each of the areas affected it is necessary to take the following actions; prediction and assessment of impact, planning of preservation measures, identification of effectiveness of preservation measures. It is important that the problems such as noise, vibration,

draught, ground subsidence, etc. be followed up for changes before and after the construction, with a comparison of status changes before and after the tunnel is put in service. The prospective scope of each impact should be investigated be continued until possible problems after starting construction and their significance can be grasped.

c. In many cases, the environment is assessed for whole projects, such as construction of roads and railways. For the sections pertaining to tunnelling in whole project, environmental preservation measures need to be reviewed, through investigation, prediction, and evaluation in a manner suitable for the purpose, utilization patterns, tunnelling method, and environmental conditions of the surroundings.

2) Research of Laws and Regulations which Regulate the Project

In planning a project, advance research shall be conducted regarding the existence of any applicable laws and regulations, the contents, procedures, measures to be taken, etc.

3) Research of Requirement Compensation

Requirements for the acquisition of the land necessary for the project shall be researched as well as those for the compensation items, including acquisition and termination of rights subsequent to the project.

Table 1-10: Environmental Investigation Items for the Tunnel Surroundings

Items	Investigation Subject	Extra Attention	Application stage of the investigation			
			Planning	Design	Construction	Maintenance
Topographic features, geological/geotechnical condition	Present condition of the landform, stratigraphy, geological structures, geological age of rocks, layer thickness, compressibility, permeability, mineral resource, distribution of natural gas	Discontinuity, faults, hazardous gas	+++	+++	+	-
Groundwater	Distribution and permeability of aquifer, groundwater pressure, and water quality, flow direction and velocity of the groundwater, distribution, and quantity of the spring water on the ground surface, recharge quantity of the groundwater.	An adequate preliminary survey is necessary. Examination of wells	+++	+++	+	-
Ground surface water	Surface water, spring, hot springs, lake, marsh, and swamp	Evaluation of water flow rate in the area of impact.	+++	+++	+	+
Ground subsidence	Annual and total subsidence of ground surface in the region of the project, multilevel subsurface settlement and settlement rate, the impact on buildings and other structures of the ground subsidence	The investigation of the construction hysteresis in a past.	+	+++	++	+
Flora and fauna	Plant and animal (especially, rare species), ecosystem investigation	The natural environment resources in which preservation is necessary	+++	+++	+	+
Land use, existing structure, cultural assets, natural monument	Land use such as land, buildings, parks, scenic district houses, and buried objects such as water supply or gas, established drift, cultural assets, natural monuments, remains, etc.	Owner and manager of land, buildings, and other properties	+++	+++	+	-
Deformation of rocks and structures	Condition of the building (structural type, soundness, application, position, etc.), landform (condition, unstable landform, physical property of the ground, etc.), Situation of the land use (application, usages, etc.), structure in the vicinity with the possibility of deformation	Investigate the situation before the tunnelling works, extra attention to shallow overburden area, landslide, and fault	+	++	+++	++
Muddy water	The status of drainage, flow rate and water quality, drainage route, and condition of waterway and river (flow rate, water quality, situation of utilization)	Follow the official regulations, investigate causes and situations of muddy waters before discharge for impact assessment	+	++	+++	++
Traffic interruption	The situation of roads and around the proposed tunnel (structure, traffic volume, congestion, road administrator, environment around the road)	Places where cross-traffic is impossible or refuge recesses.	+	++	+++	++
Shortage of water	Situation (application, use), groundwater level, water quality (water temperature, turbidity, dissolved component, odour, colour, etc.), situation of water source (type, feed rate, supply route, variation, etc.), Construction in the vicinity with the possibility of causing a water shortage	Aquifer, impermeable layer, Monitoring of groundwater level before the commencement of the tunnelling works	+	++	+++	++
Noise, vibration	Background noise, background vibration, topographic features and geology (overburden and physical property of ground, etc.), land use (application, distribution of houses and facilities which are affected by the noise and vibration from the tunnelling works)	Extra attention to hard rock ground near urban district, and of tunnel portal and shallow overburden area	+	++	+++	++
Dissemination, heavy metal pollution	pH, electric conductivity, water analysis, content test, and dissolution test (H_2O, H_2O_2) of spring water.	Extra attention to mineralization belt, ore deposit, Consider the control of muck and water from the tunnelling works	+	+	+++	++
Air pollution	Air contaminants (Concentration distribution, especially CO, NO_2), meteorological situations around the proposed tunnel.	Extra attention to the proposed tunnel portals and ventilation shafts	+	++	++	+++

+++: Important item to be implemented ++: To be implemented +: To be implemented as required.

CHAPTER 2 BASIC DESIGN

2.1 Basic of Design

2.1.1 General

(1) Basic of Design

The tunnel should be appropriately designed through various design conditions and items that are required from the purpose of the tunnel, location, and environmental conditions, making a comprehensive evaluation of safety, economic efficiency, workability, endurance, and maintenance in service. The tunnel is also designed taking into account the maintenance of tunnels in service and lifecycle cost.

1) Design conditions
The following design conditions should be considered.
   a. Rock properties
      Strength, deformation characteristics of rocks, stability of tunnel face, water inflow, earth pressure, etc.

   b. Impact on the surrounding environment (refer to Impact on Surrounding Structures and Environment)
      Noise and vibration, traffic interruption, decline of water level in rivers, streams, and groundwater, changes and pollution in water quality, ground subsidence and uplift, effects on structures in the vicinity, effects the existence of a portal may exert on the surrounding landscape, effects on the flora and fauna, and of the impact on the surrounding. etc.

   c. Effects of any structure in the vicinity on the tunnelling project
      The following cases should be examined with special care;
      - The tunnelling site involves in multiple tunnelling projects
      - The tunnel in service may be subject to the effect of any other construction in the vicinity
      - The urban tunnel with shallow overburden has the likelihood to affect the area in the vicinity.

   d. Effects of earthquakes, earth pressure, and other significant phenomena
      Water pressure (inner water pressure, external water pressure), groundwater table, water inflow amount, frost heaving and freezing, inner load, surcharge, deadweight environment (water quality, landscape), etc.

2) Design items
Major design items include the following.
   a. Horizontal alignment: Horizontal Alignment of Tunnels
   b. Longitudinal profile: Vertical Alignment of Tunnels
   c. Tunnel shape
   d. Tunnel Facilities
   e. Rock mass classification
   f. Shape of tunnel drive

g. Excavation method
h. Tunnel driving method
i. Tunnel support systems (shotcrete, rock bolts, steel ribs, etc.)
j. Lining
k. Invert
l. Waterproofing and drainage
m. Portal section and portal
n. Reinforcement tunnel support systems
o. Monitoring

In addition to the above items, some design items have special conditions, for example, branching sections and enlargement sections, as well as the design of protection against other construction works in the vicinity.

(2) Design Method

A design method shall be selected with sufficient consideration for the requirement that it match the given design conditions and the required design items. Designing a mountain tunnel, except for cases with special conditions, involves difficulties in assuming the working loads on the tunnel concerned, based upon the design conditions of a particular tunnel. Therefore, each mountain tunnel project uses a specific design technique focusing on past designs and work experiences.
Tunnel design methods are classified in the following ways.

1) Standard design
The standard design applies to the tunnelling projects whose design conditions are not significantly different from the past designs and past construction results. Each contractor uses its own design standard where the standard tunnel support system is set according to the class of rock attributed to the tunnelling site, making it possible to uniquely determine a certain tunnel support system once the class of rock and other design conditions are determined.

2) Design based on similar conditions
The similarity design method is used frequently if the tunnelling project concerned has quasi-similarity to past tunnelling projects in purpose and specifications. Even if the purpose and specifications are different, the use of the data on past tunnelling projects is effective for designing, on the condition that there is a similarity in topographic and geological conditions.

3) Design by analytical method
Analytical methods are used for projects where design study cannot expand with standard design methods and similarity design methods only because the site involves special design requirements (for example proximity of existing structures, shallow overburden, suffering a significant deformation, and earth pressure, large tunnel). These analytical procedures are being widely used with noticeable improvement in numerical calculation techniques. However, calculation results by analytical method may vary largely depending upon analytical conditions (setting a model, boundary conditions, and physical values of rock input). For this procedure, sufficient attention should be paid to the setting of analytical conditions and evaluation of results. At the same time, it should be noted that analytical techniques cannot be applied to all kinds of rocks, especially for the instability and discontinuity of the rocks.

(3) Design Procedure

This standard specification classifies tunnels into two large categories, i.e., original design and modified design.

- Original design: The design based on the plan and investigation in the stage prior to the start of construction. The term 'preliminary design' is also used.
- Modified design: The reviewed and revised 'original design' based on the visual inspection and monitoring during the construction stage.

To optimize cost-effectiveness, working efficiency, and safety, it is important to properly validate the design and to make a necessary modification of the design during tunnelling works. Qualitative procedures for modification of design include a review of design conditions, work conditions, supports and reinforcement works, and rock stability. Quantitative procedures include establishment, in advance, of tunnel drivage methods on the basis of the specified control levels utilizing monitoring results and methods utilizing face assessment scores that are given by quantifying face mapping results.

(4) Modification of Design

The original design is made based on planning and investigation prior to the commencement of the tunnel drivage. The modified design is established by review and modification of the original design based on the results of visual inspection on the rocks and monitoring during tunnelling. When designing a tunnel, evaluation of the supporting function of the rocks is of vital importance. It is necessary to modify the design by daily reviewing the rock mass classification during tunnelling. Modification of design include:

i) change of tunnel supports based on reviewing the rock mass classification,

ii) change of tunnelling method based on re-evaluation of rock conditions, and

iii) modification of design as a result of reviewing the basic planning such as tunnel alignment.

Since the tunnels are underground long linear structures, it is difficult to investigate geological conditions precisely over the whole alignment. The original design is just a design prepared through evaluation and assumption of the rocks based on limited investigation results. In order to modify the design without delay, it is necessary to compile and arrange specific design conditions and examination items during the construction phase. When preparing the modified design, it is essential to comprehensively evaluate and study information and data during the design and construction phases. Previous tunnel designs and work records under similar earth conditions are an excellent reference for modifying the design. In order to keep the project moving forward, it is necessary to change the design immediately whenever necessary.

2.1.2 Selection of Tunnel Shape and Tunnelling Method

(1) Tunnel Shape

The tunnel shape should be determined taking into full consideration not only the required inner shape, supports, lining, drainage, thermal insulation, convergence after excavation but also the long-term stability of the tunnel. In bad rock conditions, the tunnel should include the invert. In rocks with a large convergence, the sectional shape should be determined considering allowable deformation.

1) Tunnel face stability

The tunnel face stability depends upon the division of heading and round length. The division of heading, tunnel shape, and round length should be therefore designed based on the face stability in consideration of the effects of reinforcement works.

The stress distribution around the tunnel is affected by tunnel shape and initial stress distribution in the earth. The sectional shape of the tunnel should be smooth to prevent concentration of stress.

The tunnel shape is especially important when the lining is so designed to support loads. A horseshoe shape is the most common shape. When a very large act thereon, placements of invert and/or adaptation of a circular shape should be studied. When placing inverts, the transitional part between side wall and invert should have a curvature as large as possible.

2) Tunnel shape in consideration of excavation method
When designing the tunnel shape, stability during excavation should be studied in addition to stability after completion of tunnelling works.
Excavation methods include (refer to Selection of Excavation Method): i) full-face excavation method, ii) bench cutting method, iii) centre diaphragm method, iv) drift advancing method, etc.

3) Location conditions and tunnel shape
In the design of tunnel shape, rock conditions and site location conditions such as existence of structures in the vicinity should be studied.
In soft rocks, it is necessary to examine the stability of the surrounding rocks and surface settlement through analytical methods, etc.

(2) Tunnel Excavation Method

1) Excavation method
The classification and characteristics of excavation methods are shown in Table 2-1. Excavation methods widely used are full-face method, bench cut method and drift advancing method. The centre diaphragm method is also used for projects of large-diameter tunnels.

2) Rock condition and excavation method
The division of heading and round length depends upon rock conditions.
Accordingly, when selecting and excavation method, tunnel face stability should be the primary concern. For a project in soft rocks, reinforcement works at the foot of side concrete and enclosure of the tunnel section by temporary invert may be installed.

3) Tunnel shape and excavation method
In small diameter tunnel projects, the full face method is usually selected because of the limitations on equipment that can be used. In large diameter tunnel projects, partial excavation such as the bench cut method is usually selected to ensure tunnel face stability and because of higher adaptability to changes in rock conditions.

4) Impact on surrounding structures and environment, and excavation method
In a project where the tunnelling is near each other structures, surface settlement, displacement of structures in the vicinity, and their allowable limits are examined in advance. In a project in soft rocks with a small burden, applicable methods, are i) side drift advancing method, ii) bench cut method and full-face method with bench cut using reinforcement works to achieve quick enclosure of the tunnel section.

(3) Tunnel Driving Method

Tunnel driving methods include drill and blast, mechanical excavation, and combinations of these methods. As a general rule, the main concern in the selection of the tunnel driving method is rock conditions. The drill and blast are mainly applied to hard to medium-hard rocks. Mechanical excavation is usually applied to medium-hard rock to soil. Excavation equipment should be properly selected in consideration of tunnel length and rock properties. TBMs can be employed for relatively long tunnels in hard to

medium-hard rocks. When the TBM method is employed, excavation efficiency and prevention measures against problems shall be studied in advance.

Table 2-1a: Classification and Characteristics of Excavation Methods

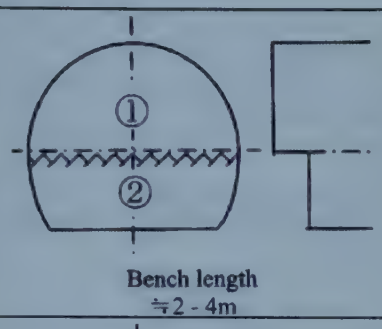
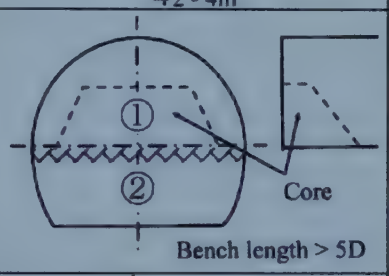
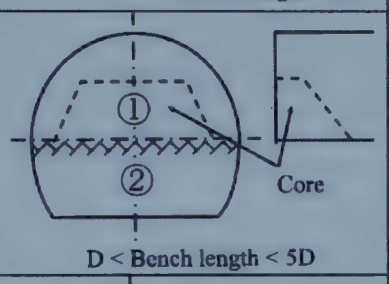
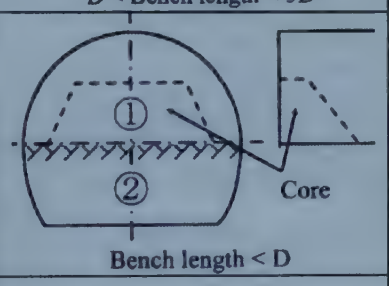
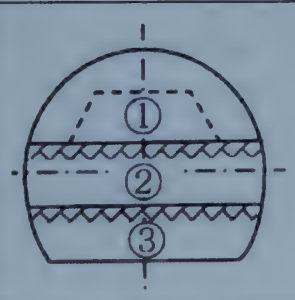
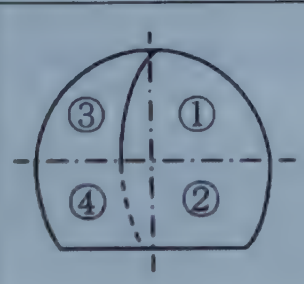
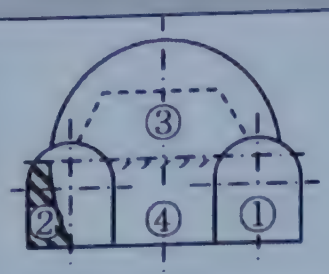
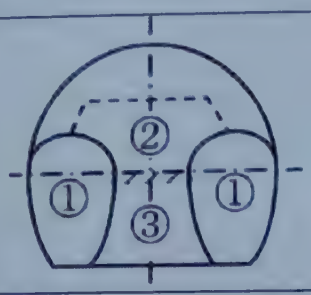
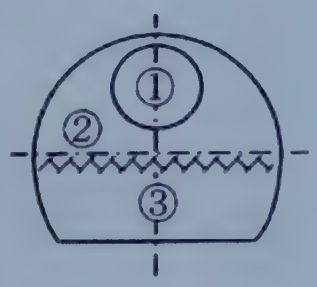
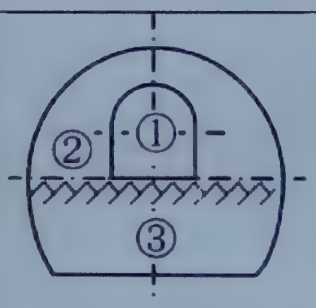
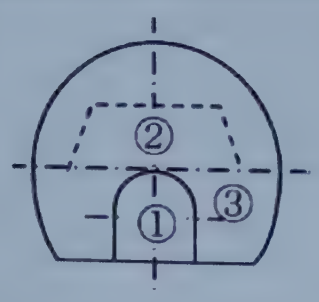
Excavation Method	Division of Heading	Applicability	Advantages	Disadvantages
Full Face Excavation		- Almost all rocks for small diameter tunnels. - Very stable rocks for large diameter tunnels ($A>60m^2$) - Fairly stable rocks for medium diameter tunnels ($A>30m^2$) - Inappropriate for good rocks interspersed with bad rocks	- Labour saving by mechanization - Work control including safety control is easier because of the single-face excavation.	- Full tunnel length cannot be by full-face alone. Bench cut could be adopted as required. - Unstable rocks from the crown may fall down with large stress in rocks around the tunnel, and additional safety measures may be required.
Full Face Excavation with Bench Cut		- Fairly stable rocks where full-face excavation is inapplicable. - In case that full-face excavation becomes difficult - For good rocks interspersed with poor rocks	- Labour saving by mechanization and parallel excavation of top heading and bench. - Work control and safety control are easy because of the single-face excavation.	- Difficult to switch to other excavation methods
Bench Cut: Long Bench Cut		- Fairly stable rocks where full-face excavation is inapplicable. - Ring cut is applicable in case the face is unstable.	- Alternate excavation of top heading and bench reduces equipment and manpower.	- Alternate excavation extends the work period.
Bench Cut: Short Bench Cut		- Ring cut is applied when the face is unstable.	- Adaptable to changes in rock conditions - Alternate excavation of top heading and bench reduces equipment and manpower.	- the work cycle of parallel excavation of top heading and bench is hard to be controlled. - Alternate excavation extends the work period.
Bench Cut: Mini Bench Cut		- In the case of convergence needs less control than the short bench cut method. - Mini bench is used to make the tunnel stable as soon as possible. - Ring cut is applicable in case the face is unstable.	- Easy to make early closure with invert. - Alternate excavation of top heading and bench reduces equipment and manpower.	-the space for heavy machines may be hard to be kept on the bench.
Bench Cut: Multiple Bench Cut		- Fairly good rocks for high and large-diameter tunnels. - Bad rock that requires a small diameter of heading to stabilize the face.	- Face can be stabilized easily.	- Large deformation may develop if the installation of a full tunnel support system is delayed. - Each bench length is limited and working space is limited. - Careful operation of mucking at each bench is required
Centre Diaphragm Method		- to minimise the ground surface settlement at the soil with shallow overburden. - Relatively bad rockss for large diameter tunnels.	- Face can be stabilized by dividing into small sections. - Ground surface settlement can be reduced. - Divided sections of heading are larger than those in the side drift method, and larger equipment can be used.	- Displacement or settlement during removal of diaphragm should be monitored. - Removal of diaphragm is included in the work process. - The adoption of a reinforced works in the tunnel is difficult.

Table 2-1b: Classification and Characteristics of Excavation Methods (continued)

Excavation Method		Division of Heading	Applicability	Advantages	Disadvantages
Side Drift Driving Method	With wall Concrete		- Rocks where bearing capacity is insufficient and bearing capacity must be improved before the excavation of top heading. - Soft rocks or soil with shallow overburden where unsymmetrical earth pressure or landslide is anticipated.	- Relatively massive concrete wall for side drift improves the bearing capacity and strengthens resistance against unsymmetrical earth pressure.	- Only small equipment can be used for drift excavation. - The upper part of the tunnel may be loosened by drift excavation.
	Without Wall Concrete		- Rocks where bearing capacity is insufficient to apply bench cut method. - Soft rocks with shallow overburden where ground surface settlement is anticipated.	- Ground surface settlement can be reduced. - Temporary diaphragm can be more easily removed than those in centre diaphragm method.	- Only small equipment can be used for drift excavation.
Centre Drift Driving Methods	Centre Top Drift Driving		- To find the conditions of rock, groundwater, and rock deformation prior to the main tunnel - to decrease the stress in rocks around the main tunnel. - TBM can be adopted to centre to drift.	- Conditions of rock, groundwater, and rock deformation can be predicted prior to the main tunnel - Stress in rocks around the main tunnel can be decreased. - Blasting is not required for excavation of the centre drift. Blasting vibration and noise can be reduced. - Face stability can be improved when enlarged - Ventilation effect can be expected when drift excavation is completed.	- Drift excavation by TBM may take time unless rocks are stable. - Small equipment has to be used for drift excavation.
	Centre Drift Driving		- To find the conditions of rock, groundwater, and rock deformation prior to the main tunnel - to decrease the stress in rocks around the main tunnel.	- Conditions of rock, groundwater, and rock deformation can be predicted prior to the main tunnel - Stress in rocks around the main tunnel can be decreased. - Blasting is not required for excavation of the centre drift. Blasting vibration and noise can be reduced. - Face stability can be improved when enlarged	- Small equipment has to be used for drift excavation.
	Centre Bottom Drift Driving		- For drainage of inflow water	- Geology can be confirmed by advancing drift. - tunnel face of the main tunnel can be produced from drift, and the construction period can be shortened.	- Difficult to balance the work cycle of each face. - Various types of equipment are required.

2.1.3 Design Conditions

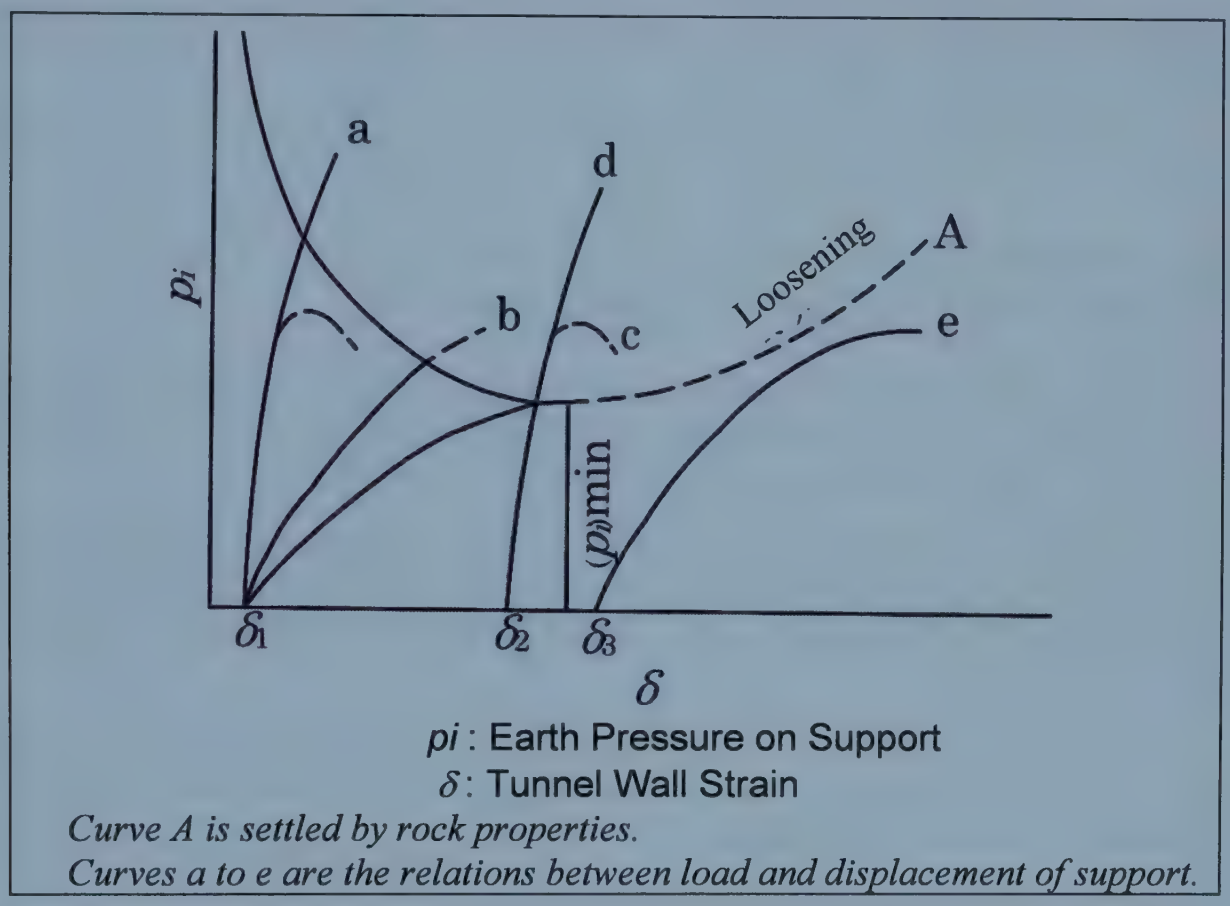
(1) Rock Properties

1) Tunnel design and rock properties

In tunnel design, it is essential to precisely evaluate rock properties in order to fully utilize the supporting function of the rocks. The results of inspection and monitoring during tunnelling should be fed back to modify the design.

2) Rock properties and design of support and lining

When applying the design based on similar conditions, applicability should be evaluated on the basis of geological conditions, seismic wave velocity, qu/P_v ratio, and rock properties. When applying analytical methods, modulus of deformation, poisson's ratio, and other rock properties are used as input data.
Earth pressure to be considered in mountain tunnels includes pressure acting on supports or lining due to soft rocks, and pressure upon supports due to displacement of the excavation surface. Loosening earth pressure, which should be considered in heavily fissured hard rocks or soil, is estimated by Terzaghi's formula or on the basis of the whole overburden load. Earth pressure working on supports, induced by displacement of excavation surface, can be expressed by Figure 2 1.
Even no abnormal behaviour is found in rocks during tunnelling, earth pressure may increase even after the tunnel is in service and result in significant deformation. Since earth pressure may act on lining in the rocks that continues to swell over a long period of time, suitable measures such as placement of invert are required.



pi : Earth Pressure on Support
δ : Tunnel Wall Strain
Curve A is settled by rock properties.
Curves a to e are the relations between load and displacement of support.

Figure 2-1: Conceptual Diagram of rock and Support Characteristics Curve (Fenner-Pacher Curve)

3) Rock properties such as tunnel face stability and water inflow
Typical rock properties affecting work efficiency are face stability and water inflow. Face stability in rock mass depends on rock strength, fissure, swelling properties, etc. In contrast, face stability in soil depends upon overburden, cohesion, grain size distribution, water content, groundwater table, etc.
Face stability is not significantly affected by water inflow in medium-hard to hard rocks, whereas it is noticeably affected in soil. Even in medium-hard to hard rocks, caution should be exercised regarding unexpected heavy inflow.

(2) Impact on Surrounding Structures and Environment

The environmental investigation items given in Table 1-10 are essential concerns in design. In urban tunnelling projects, tunnels are often planned in locations where there are important structures and buried objects nearby. Settlement may include not only that during excavation but also continued consolidation settlement after completion of the tunnel. Impact on the surroundings should be therefore one of the main concerns.

2.2 Rock Mass Classification

2.2.1 Rock Mass Classification

(1) Rock Mass Classification for Road Tunnel (in Japan)

This criterion classifies rocks based on quantitative indicators. Rock mass classification based on classification criteria is being utilized effectively for purposes such as an index to designate rock conditions for tendering and awarding contracts. The criteria for classification of rocks, typically expressed as a class of rocks, should meet the following conditions.

- The classification factor shall be objective and quantitative
- The classification factor shall be related to the purpose and design of the tunnel.
- The classification factor shall be standardized over the total length of the tunnelling lot concerned.
- The classification factor shall be able to be obtained through an ordinary investigation method and test method.
- The criterion used in classification (class of rocks) shall correspond to the design and work lot.

The rock mass classification criteria used in road tunnels is shown in Table 2 2.
Factors which are applying to Table 2 2 are described below.

1) The rock mass classification table is designed to be used for planning of two-lane tunnels with an overburden thickness of more than 20 meters but less than 500 meters, but may apply for three-lane tunnels.
2) In this table, a standard classification cannot be applied to special cases such as ground in the proximity of a portal where there is localized earth pressure, the likelihood of landslide, and the case involving the necessity of confining land subsidence.
3) The use of a class of rock "E" should be limited to the sites with 200 mm or more convergence with special lithological character (rocks in which earth pressure is significant because of large talus and large fault fracture zones).
4) Rock mass classification criteria
   In the process of the original design, rocks should be classified from a comprehensive viewpoint, putting together all related results of investigations (surface geological investigation, geotechnical boring, rock samples test, etc.). Since the two parameters of seismic wave velocity and qu/P_v ratio are effective as a rough standard, their role is limited to a complement to the surface geological investigation, geotechnical boring, and laboratory rock tests. These three investigations are important for rock mass classification in the initial stage.

Table 2-2: Rock mass classification

Class of ground	Rock type	Representative rock name	Elastic wave velocity (Vp km/s) 10 20 30 40	Geological condition: Effect by water and lithologic character	Geological condition: Interval of discontinuity	Geological condition: Condition of discontinuity	Boring core condition RQD	Ground competence factor	Situation of the tunneling and standard of the displacement
B	H Massive	Granite, granodiorite, quartz porphyry, hornfels		- Rock is extremely hard and fresh or a slight tendency toward deterioration due to weathering action is to be seen - No deterioration due to water.	- The interval of the joint is about average 50cm. - Though there is the influence of bedding and schistosity, these influence on the tunneling has been limited	- It has almost no slickenside and fault clay in discontinuity - Discontinuity has closed approximately.	- The shape of the core shows large pieces, short cylinders or rods, core length being generally in rang 10-20cm, but with length on order of 5cm also present - RQD is over 70	———	- The rock strength is very bigger than the load which is caused by the tunneling. - The condition of discontinuity is good, and the looseness by the tunneling does not occur almost - Rock falls from the excavation face occur rarely, and the convergence with the excavation is elastic deformation of about 15mm or less - Cutting face stands up.
		Paleozoic and Mesozoic sandstone,chert							
	M Massive	Andesite, basalt, rhyolite, dacite							
		Tertiary sandstone or conglomerate							
	L Massive	Serpentinite, tuff, tuff breccia							
	M Layered	Slate, Paleozoic and Mesozoic shale							
	L Layered	Black schist, greenshist							
		Tertiary mudstone							
C I	H Massive	Granite, granodiorite quartz porphyry, hornfels		- Rock is relatively hard and fresh or a slight tendency toward deterioration due to weathering action is to be seen. - Soft rock which is relatively consolidated - No deterioration due to water.	- The interval of the joint is about average 30cm - Bedding and schistosity are remarkable, and these affect the tunneling	- It has a few slickenside and fault clay in discontinuity - Though discontinuity is partially opening, the width is small	- Core length being generally in rang 5-20cm, but with length on 5cm or less also present. - RQD is 40-70	——— Over 4	- The rock strength is bigger than the load which is caused by the tunneling - The condition of discontinuity is good, and the looseness by the tunneling occur partly - The partial rock falls along discontinuity which is easy to slide relatively occur rarely, and the convergence with the excavation is elastic deformation of about 15～20mm - Cutting face stands up
		Paleozoic and Mesozoic sandstone,chert							
	M Massive	Andesite, basalt, rhyolite, dacite							
		Tertiary sandstone or conglomerate							
	L Massive	Serpentinite, tuff, tuff breccia							
	M Layered	Slate, Paleozoic and Mesozoic shale							
	L Layered	Black schist, greenshist							
		Tertiary mudstone							
C II	H Massive	Granite, granodiorite quartz porphyry, hornfels		- Rock is relatively hard and fresh or a slight tendency toward deterioration due to weathering action is to be seen. - Rock becames somewhat soft as result of weathering and alteration. - Soft rock which is relatively consolidated - It has some deterioration or looseness due to water	- The interval of the joint is about average 20cm - Bedding and schistosity are remarkable, and these affect the tunneling.	- It has some slickenside and fault clay in discontinuity - Opened discontinuity increases, and the width of discontinuity also expands - The rock with small fault which is narrow width.	- Core length being generally in 10cm or less, and with numerous pieces of 5cm or less. - RQD is 10-40	——— Over 4	- Though the rock strength is not very bigger than the load which is caused by the tunneling,it is in the range of the plastic deformation - Sincet the condition of discontinuity is bad, even if the strength of rock is big, rock block intend to fall along the discontinuity which is easy to slide and the looseness by the tunneling increase - When strength of rock is smaller than the load which affects it, the convergence with the excavation achieved at about 30mm which is the elasto-plasticity boundary but it is almost settled, until the face separates on the displacement 2 times of the diameter of the tunnel(2D) - Cutting face stands up.
		Paleozoic and Mesozoic sandstone,chert							
	M Massive	Andesite, basalt, rhyolite, dacite							
		Tertiary sandstone or conglomerate							
	L Massive	Serpentinite, tuff, tuff breccia							
	M Layered	Slate, Paleozoic and Mesozoic shale							
	L Layered	Black schist, greenshist							
		Tertiary mudstone							
D I	H Massive	Granite, granodiorite, quartz porphyry, hornfels		- Though the hard rock remains partly, it received generally strong weathering and alteration. - Bedding and schistosity are remarkable. - The interval of the discontinuity is about average 10cm or less, and most of their are opened - Width of discontinuity big and most of ther have been slickenside and fault clay. - The rock within small fault which is narrow width and with clay. - Soil material mixed numerous pumice, talus, etc - It has remarkable deterioration or looseness due to water.			- Cores take form of small pieces, but sometimes form of clay or sand with rock fragments mixed in. - RQD is 10 or less.	4-2	- The rock strength is bigger than the load which is caused by the tunneling,it is occur the plastic deformation and elastic deformation partially - Sincet the condition of discontinuity is very bad, even if it is sufficient in the strength of rock fixing the plastic deformation, the looseness by the tunneling is incressed along many discontinuity which is easy to slide. - When strength of rock is smaller than the load which effects it, the convergence with the excavation achieved at about 30-60mm without the case which carried out the invert closing early, and it is almost settled, until the face separates on the displacement 2 times of the diameter of the tunnel(2D) - The face is unstable, and ring cuts and face shotcrete are necessary by the condition of the ground
		Paleozoic and Mesozoic sandstone,chert							
	M Massive	Andesite, basalt, rhyolite, dacite							
		Tertiary sandstone or conglomerate							
	L Massive	Serpentinite, tuff, tuff breccia							
	M Layered	Slate, Paleozoic and Mesozoic shale							
	L Layered	Black schist, greenshist							
		Tertiary mudstone							
D II	H Massive	Granite, granodiorite quartz porphyry, hornfels						2-1	- Though the rock strength is smaller than the load which is caused by the tunneling,it is occur the plastic deformation and large elastic deformation - Sincet the rock strength is small and condition of discontinuity is very bad, the looseness by the tunneling expands along many discontinuity which is easy to slide, and the displacement also increases.The convergence with the excavation achieved at about 60-200mm without the case which carried out the invert closing early, and it is not settled, even if the face separates on the displacement 2 times of the diameter of the tunnel(2D) - The face is unstable, and ring cuts and face shotcrete are necessary by the condition of the ground
		Paleozoic and Mesozoic sandstone,chert							
	M Massive	Andesite, basalt, rhyolite, dacite							
		Tertiary sandstone or conglomerate							
	L Massive	Serpentinite, tuff, tuff breccia							
	M Layered	Slate, Paleozoic and Mesozoic shale							
	L Layered	Black schist, greenshist							
		Tertiary mudstone							

Note 1) Rock class A is supposed to be good rock which does not correspond to the table, rock class E is supposed to be inferior (convergence, over 200mm)

Note 2) The division of H,M,L. By the strength in the primary and fresh condition of rock, it will be divided at the uniaxial compressive strength as following

H: $q_u \geq 80N/mm^2$, M: $20N/mm^2 \leq q_u < 80N/mm^2$, L: $q_u < 20N/mm^2$

Note 3) The division of Massive Layerd Massive: The rock which the joint plane becomes dominant surface of discontinuity
Layerd: The rock which bedding plane or schistosity plane become dominant surface of discontinuity

Note 4) Convergence means the change of the distance between tunnels wall surface measured under tunneling actually, and it is not contained the displacement before the excavation

Note 5) Looseness means that the rock block field intends to fall along discontinuity by the gravity, since discontinuity in rock mass which has closed by earth pressure opened because of the release of in-situ stress by the tunneling

Note 6) The strength of rock means the strength of the rock which does not receive the influence of the fissure

(2) Seismic Wave Velocity

When investigating the rocks for tunnelling, to make up for the lack of data, the use of geophysical surveys, such as seismic surveys, is recommended as a complementary means to understand the characteristics of the rocks. Since the tunnel is a long structure and deep in the ground, and tunnelling may accompany some ground sections or places where for ground observation, it is absolutely impossible to observe the rock condition without geotechnical borings. The seismic survey is effective in the observation of physical properties of rocks over a wide range, and a comparatively simple approach. However, this approach is nothing more than an indirect means with the risk of a substantial margin of error.

1) The effectiveness of a seismic survey is practically limited to approximately 100m in depth, because the required number of surveying lines is 5 to 6 times the survey depth, assuming that the hardness increases from the surface to the depth (in other words, seismic wave velocity increases). This approach cannot be used for cases where inverse reactions may arise, with a hard layer in the depth.

2) In cases where the earth has excess earth stress due to folds in the presence of shale, slate, and schist or in cases where rocks, involving many minor cracks, is likely to loosen, the class of the for this site may be assessed one rank higher by the initial grade given by seismic survey.

3) When the seismic wave velocity (P wave velocity) and the value of the qu/P_v ratio fall into two classes, the evaluation shall be based on topographical characteristics, rock conditions, etc.

4) Near portals and swift streams, the overburden depth above and to the side of a tunnel is often small. In such cases, attention shall be paid to the seismic wave velocity and the class of rocks shown in this table shall be lowered if necessary.

5) If the overburdened earth in the range of approximately 15m upward from the proposed elevation of the tunnel consists of strata having more than a single velocity, the velocity of the stratum above the planned elevation which has the slowest velocity in the seismic wave velocity distribution shall be adopted.

6) Particular care is required in locations where the overburden depth is shallow, because at such sites rock conditions often vary markedly with the result that errors in surveying and in the analysis of geophysical survey results have a great influences on the evaluation of rock mass classification.

7) Regarding faults and fracture zones, other criteria, such as their direction and the overburden depth, shall be considered in addition to the seismic wave velocity.

8) When the seismic wave velocity has been obtained within the tunnel during excavation, this shall be used to confirm the rock mass classification and to change the prior design.

(3) Status of General Rocks

1) Lithology

The lithology referred to here is a group of rock fragments composing the current rock mass that once was a new geological mass, but subsequently deteriorated through weathering. In the preliminary investigation, the effort focuses on directly and quantitatively understanding the strength of rocks, through laboratory tests of samples from boring cores. During tunnelling, it is necessary to use unconfined comprehensive strength test and point loading test to assess the rocks, including a hammering test to complement the data.

2) Effects of water

The rock strength may deteriorate under the effects of groundwater, so its significance should be considered the structure of a tunnel and the difficulty in excavation. If tunnelling site may involve water influxes, the process of original design should, for ground assessment. assume the presence of rock weathering due to groundwater, and during the tunnelling process, the assessment of rock qualities should be corrected considering the actual amount of inflows and the degree of rock weathering.

(4) Status of Discontinuities

If the ground constitutes a discontinuity that governs the behaviour of a tunnel, this item is given top priority in-ground assessment. In other words, the shear strength of rock mass is determined by the geometry of a discontinuity and the kind of substance material interspacing the surfaces of discontinuities. Therefore, assessment should be made by grasping the behaviour of rock mass, considering as the main factor the roughness of a discontinuity (geometry and surface sliding) and filling material such as clay, as well as evaluating the length (continuity), the width (gap) and weathering status comprehensively. The process of preliminary investigation should integrate as much direct observation as possible through ground surface geological investigation and core boring. During the process of tunnelling, it is possible to grasp the status of discontinuities by looking at the details of the cutting face.

(5) Gaps between Discontinuity

The gap between discontinuity surfaces is represented by a regularly developing cracking line in bedding, schistosity, and joint. For a tunnelling project, this factor needs to be evaluated about the importance if the crack may produce pronounced irregularity on the cutting face as well as about the risk that some blocks may be separated and fall down because of the gap. The process of a preliminary investigation, for this matter, should integrate as much direct observation as possible through ground surface geological investigation and core boring investigation. D During the tunnelling, it is possible to grasp the status of gaps between discontinuities by observing the details of the cutting face.

(6) Boring Cores

In the preliminary stage of design, boring cores are a important approach to confirm the rock condition directly. The results from the survey of boring cores are used as part of the ground surface investigation, especially for evaluating the strength of rock fragments, the status of discontinuity, and the importance of gaps. The status of boring cores, although difficult to use as a standard judgement criterion because RQD is affected by core diameter and skill of boring operation, is able to be a standard for rough judgement. This standard applies to the core samples taken by a double core tube with a 66mm outer diameter of boring.

(7) qu/P_v Ratio

The qu/P_v ratio is defined as follows:

$$qu/P_v \text{ ratio} = q_u / (\gamma \cdot H)$$

Where q_u = unconfined comprehensive strength of rock (tf/m^2)
γ = unit weight of the rock (tf/m^2)
H = overburden depth (m)

For rocks in which the presence of fractures, etc., can be ignored, the unconfined comprehensive strength of specimen can be adopted as the unconfined comprehensive

strength of the rock, but for rock in which the effects of fractures, etc., is large, the quasi-strength of rock mass (tf/m^2) shall be used.

$$q_u' = (V_p / U_p)^2 \cdot q_u$$

Where V_p = seismic wave velocity of rock (P wave, km/s)
U_p = ultrasonic wave velocity of specimen (P wave, km/s)
q_u = unconfined comprehensive strength of specimen (tf/m^2)

In general, U_p is equal to or greater than V_p, but if V_p exceeds U_p, the quasi-strength of rock mass shall be found by using V_p for U_p.

(8) Tunnelling and Ground Displacement Monitoring

Ground displacement should be monitored as soon as possible after muck handling (at least. 3 hours or less). The grade of assessment once attributed to the ground concerned may be corrected, if needed, taking into account the relationship between the tunnel axis and direction/inclination of discontinuity in the cutting face during the excavation phase.

(9) Kinds of Rock Needing Attention

The kinds of rock shown below need to be monitored since they frequently cause problems in tunnelling. They may be ranked lower than the original grade in some cases.

a. Serpentine and rock which has been subjected to serpentinization, mudstone, shale, tuff, and pyroclastic deposits easily deteriorate due to water and therefore require thorough attention.

b. Because the weathering of serpentine is extremely irregular, the actual condition of the rock often cannot be observed from the results of the geophysical survey. Therefore, thorough attention is required in the construction stage.

c. Because of diabase. amphibolite, peridotite, gabbro, and shalistein are subject of serpentinization, they require the same attention as serpentinite.

d. Serpentinite, propylite, black schist, mudstone, and tuff shall be reduced to class "D_{II}" or "E" if swelling is clearly confirmed.

2.2.2 Rock Mass Classification for Ground Fluid

(1) Empirical Method

Items shown below are among the rock conditions affecting the level of difficulty in the excavation of tunnels, which are in the categories of rock mass classification and physical properties, and which are difficult to clearly define. When defining such rock conditions, it is very important to qualitatively assess and evaluate based on past experience and reference documents from previous tunnelling sites.

- Stability of face
- Unexpected heavy inflows
- Topography with the risk of localizing pressure
- Collapse and landslide in the proximity of the portal
- Effects on the buildings in the vicinity
- Presence of obstacles in rocks such as fault, swelling and rock burst and find their characteristics

Soil and fault fracture zone need in-depth study regarding the likelihood that the face would become extremely unstable or collapse due to unexpected heavy inflows. Table 2-3 shows the examples of indicators in judging whether or not the ground will become fluid.

Judging whether the rocks concerned is likely to swell or not is an important step in determining the tunnel geometry, excavation method, reinforcement works, and studying post-loading. Table 2-4 shows examples of swelling indices.

Table 2-3: Example of Indices which Indicate the Possibility of the Ground becoming Fluid

	Index	proposer
a	– Unit weight $\leqq$ 2.65 g/cm^3	Yabe, et al (1969)
	– Specific gravity of soil particles $\leqq$ 1.7	
	– Uniformity coefficient $\leqq$ 4 50%	
	– Particle size $\leqq$ 1.55 mm	
	– 10% particle size $\leqq$ 0.15mm	
b	– Fine fraction content $\leqq$ 10%	Morito (1973)
c	1) Homogeneous sand layer	Japanese National
	- Fine fraction content $\leqq$ 10%	Railroads Structural
	- Uniformity coefficient $\leqq$ 5	Design office (1977)
	- Saturated sand	
	2) Sand of gravel with high groundwater level	
	3) Sand aquifer between impermeable layers	
d	– Content of cohesive soil $\leqq$ 10%	JSCE (1977)
	– Uniformity coefficient $\leqq$ 4	
e	– Content of cohesive soil $\leqq$ 8%	Okuzono, et al (1982)
	– Uniformity coefficient $\leqq$ 6	
	– Coefficient of permeability $\geqq 10^{-3}$ cm/s.	
f	1) Unstable ground	Kiya, et al (1993)
	- Relative density < 80%	
	- Hydraulic gradient near the face is large.	
	2) Ground fluid anticipated	
	- Content of cohesive soil $\leqq$ 10%	

(2) Assessment of Ground by Geotechnical Model

If designing a tunnel needs a conservative approach considering the behaviour of the ground in which the tunnel is constructed, theoretical and numerical analysis of ground behaviour and stability are implemented by providing a geotechnical model for the ground concerned. In order to determine the physical properties as input values, the borehole load test or laboratory test is used. However, the physical property values which have been determined in-situ or at laboratory tests, in many cases, are not able to represent the actual characteristics of the whole ground because of non-conformity of the ground, discontinuity due to the presence of joints, anisotropy, and size effects, etc. If these values are used without any correction, it is frequently known that the

analysis results do not equal the measurement values taken during tunnelling. Considering this, the current practice is that the physical property values to be input for analysis are determined using any of the following approaches;

- The test-base physical property is reduced proportionally with the importance of fissures (use of fissure factor)
- The results, which are obtained through measurement-based inverse analysis, are used as reference.
- Since each authority and organization concerned has its own standard physical properties, it is recommended to use the values they provide.

Table 2-4: Swelling Indices

	Swelling Index	Geology	Proposer
a	1) Gn ≦ 2 : Squeezing or swelling 2) 2<Gn≦4 : Strong squeezing or high earth pressure is expected 3) 4<Gn≦6 : High earth pressure is expected 4) 6<Gn≦10 : Some earth pressure is expected *(Gn = σc / γH : σc= unconfined compressive strength, γ = Unit weight, H = Overburden depth)*	Neogene mudstone	*Nakano (1975)*
b	>Swelling Rocks 1) Rocks contain montmorillonite 2) Content of particles of 2μm or lesser size ≧ 30% 3) Plastic index ≧ 70% 4) Cation exchange capacity ≧ 35meq/100g 5) Degree of slaking D 6) Most of boring core is fractured >Possibility of Swelling 7) Rock contain montmorillonite 8) Content of particles of 2μm or lesser size ≧ 20% 9) Plastic size ≧ 25% 10) Cation exchange capacity ≧ 20meq/100g	Neogene-Miocene Quaternary – Pleistocene	*Japan railroad Construction Public Corporation (1977)*
c	1) Elastic modulus ≦ 800 MP_a 2) Unconfined compressive strength ≦ 4 MP_a /cm^2 3) Unit Weight ≦ 2.05 gf/cm^3 4) Natural water content ≧ 20% 5) Liquid limit ≧ 100% 6) Plastic index ≧ 70% 7) Content of particles of 2μm or lesser size ≧30%. 8) Cation exchange capacity ≧ 35meq/100g 9) Swelling ratio ≧ 20%	Neogene – Miocene Quaternary	*Otsuka, et al (1980)*
d	1) Natural water content ≧ 20% 2) Unit weight (dry) ≦ 1.8 gf/cm^3 3) Absorption in first test/natural water content ≦ 2.0 4) Degree of slaking C-D 5) Montmorillonite ≧ 30% 6) RQD ≦ 30%	Neogene – Miocene mudstone.	*Sato, et al (1980)*
e	Strong Swelling Rocks 1) Rock core samples show swelling (rock core sampling must be in a dry condition) 2) Gn ≦ 1.5 (Marked if <0.5) 3) Montmorillonite content ≧ 20% and natural water content ≧ 20% 4) Degree of slaking D Swelling Rocks 1) RQD is low 2) 1.5 ≦ Gn<2.0 3) Montmorillonite content ≧ 20% or natural water content ≧ 20%.	Neogene mudstone.	*Yoshikawa, et al (1988)*

Note: 1 Kgf/cm^2 = 0.1 MP_a

2.3 Tunnel Supports

2.3.1 Design Concept for Tunnel Supports

(1) Design Concept for Tunnel Supports

Tunnel supports shall effectively utilize the inherent supporting function of the surrounding rocks. In order to effectively design tunnel supports, it is necessary to evaluate all design conditions such as topography, geology, rock properties, overburden, possibility of water inflow, tunnel cross-section, restriction of surface settlement, tunnelling methods, etc. Since the tunnel is an underground linear structure spanning a certain distance, it is generally difficult and requires much cost and time to correctly grasp the rock conditions before excavation. It is, therefore, necessary to plan and implement suitable geological/geotechnical investigation that ensures cost-effectiveness, and to introduce visual inspection methods during the construction phase. Determination of tunnel support systems includes original design based on geological investigation results and the design modified during excavation. Tunnel support systems in the original design are determined by a combination of suitable support members, based on rock mass classification categorized through geological investigation. In modified design, the suitability of the tunnel support systems in the original design is reviewed through results of visual inspection on the rocks and monitoring during excavation, and modified tunnel support systems are established if necessary.Table2-5 shows the criteria for the selection of tunnel support system. When modifying the tunnel support system in response to the classification of rocks change, it is desirable to suddenly change the supporting functions. The tunnel support system is determined by 1) application of standard design, 2) application of the design based on similar conditions, or 3) application of analytical methods. A high level of engineering judgement is required on the basis of the past tunnel projects to select the appropriate tunnel support system.

1) Application of standard design
The standard tunnel support system for each rock mass classification established on the basis of data accumulated from past tunnel projects is applied to the standard design of the tunnel support system for a proposed tunnelling project.
Tables 2-6 to 2-11 show examples of standard tunnel support systems for road tunnels.

2) Application of the design based on similar conditions
If the rock conditions and tunnel shape for a new tunnelling project are found to be similar to past tunnel projects, tunnel support systems can be determined, regardless of the intended use of the tunnel, referring to the design of the similar tunnel. In these cases, it is necessary to review applicability and to suitably modify the tunnel support system of past tunnel projects. When applying a design based on similar conditions, it is important to collect as much information as possible about rocks conditions, tunnelling methods, and monitoring results from past tunnel projects, and to analyze and evaluate these kinds of information with the results from past tunnel projects, and to analyse and evaluate these kinds of information with the results of the preliminary survey.

3) Application of analytical methods
When large pressure or displacement is anticipated, or very few similar designs are found among past tunnel projects because of a unique tunnel shape, determination of the tunnel support system in the manner 1) or 2) above may be difficult. In these cases, the tunnel support system is determined by the application of analytical methods.
When settlement should be minimized in the ground with shallow overburden, tunnel support may be quantitatively evaluated through structural analysis. In these

cases, it is essential to correctly assess rock properties and load acting on supports based on preliminary survey results, and to determine suitable tunnel support systems through analytical methods in order to satisfy the control criteria. Analytical methods include numerical analysis and theoretical analysis. Numerical analysis such as the infinite element method is capable of grasping stress distribution and deformation status in-ground and supports. Theoretical analysis, based on elasto-plastic theory, etc., can provide stress distribution by modelling tunnel a cross-section as a simplified shape such as a circle. Both analytical methods are effective in that they enable quantitative design. However, definite methods have not been established completely yet on modelling of support members and reinforcement works, effects of groundwater, rock properties, modelling of initial conditions and discontinuities of rocks, and evaluation of stress release rate. It is therefore essential to survey and examine adequately before analysing and to keep in mind the current status of the analytical method when utilizing analytical results.

Table 2-5: Criteria for Selection of Tunnel Support System

Earth Category		Support Members				Note
		Shotcrete	Rock Bolts	Steel Ribs	Invert	
Hard Rock	Sparsely Fissured	+	+	-	-	
	Heavily Fissured	++	++	+	-	
Soft Rock	Large qu/P_v ratio	++	++	-	+	Invert is necessary for mudstone.
	Small qu/P_v ratio	++	++	++	++	Early placement of invert
Soil	(Shallow overburden)	++	+	++	++	Lining may be redesigned
	(Deep Overburden)	++	++	++	++	Quick complete enclosure by tunnel support system and estimation of allowable deformation
Swelling Rock		++	++	++	++	Quick complete enclosure by tunnel support system Estimation of allowable deformation and redesign of tunnel support system

++: Very Effective, +: Effective, -: Unnecessary in principle

Table 2-6: Standard Tunnel Support system for Road Tunnels (Small Diameter Tunnels)

(Small-diameter tunnels 3 to 5 m)

Rock Mass Classification	1 cycle driving progress (m)	Rock Bolts				Steel ribs		Thickness of shotcrete (cm)	Thickness of lining[1] (cm)	Excavation method
		Length (m)	Spacing		Installation direction	Type	Spacing (m)			
			Perpendicular to tunnel direction (m)	Longitudinal direction (m)						
B	2.0	None	--	--	--	None	--	5	20	Full face excavation
CI	1.5	2.0	1.2	1.2 - 1.5	Upper & side	None	--	5	20	
CII	1.2									
DI	1.0	2.0	1.0	1.0	Upper & side	H-100	1.0	10	20	
DII	1.0	2.0 - 3.0	1.0	1.0	Upper & side	H-100	1.0	10 - 12	20	

Lining can be omitted taking into full account the service status of the tunnel ground conditions

Table 2-7: Standard Tunnel Support System for Road Tunnels (Medium Diameter Tunnels)

(Ordinary Diameter Tunnels, inner diameter: about 8.5m to 12.5m)

Rock mass Classification	1 Cycle Driving Progress (m)	Rock Bolt				Steel ribs			Thickness of Shotcrete (cm)	Thickness of lining		Allowable Deformation (cm)	Excavation method
		Length	Spacing		Installation Direction	Type	Bench	Spacing (m)		Side Wall (cm)	Invert (cm)		
			Perpendicular to tunnel Direction (m)	Longitudinal Direction (m)									
B	2.0	3.0	1.5	2.0	Top Heading 120°.	-	-	-	5	30	0	0	Full Face method with mini bench cut, or short bench cut
CI	1.5	3.0	1.5	1.5	Top Heading	-	-	-	10	30	(40)	0	
CII	1.2	3.0	1.5	1.2	Top Heading	-	-	-	10	30	(40)	0	
					Bench	H-125	-	1.2					
DI	1.0	3.0	1.2	1.0	Top Heading	H-125	H-125	1.0	15	30	45	0	
		4.0			Bench								
DII	≤ 1.0	4.0	1.2	≤ 1.0	Top Heading Bench	H-150	H-150	≤ 1.0	20	30	50	10	

1) For class A and E rocks, specifications should be determined based on the actual ground conditions.
2) As a general rule, the support pattern "b" is applied for class C Ⅱ and DI rocks, whereas application of support pattern "a" is considered when it is anticipated that the displacement induced by tunnel excavation is small and the tunnel face is stabilized
3) On invert concrete
   i) When rocks categorized as class CI and CII is Tertiary mudstone, tuff, serpentinite, or other rocks prone to argillizatlon, or weathered crystalline schist, or solifataric clay, invert concrete, the thickness of which is given in parentheses should be placed.
   ii) When early closure by tunnel support system is necessary, shotcrete is applied as the invert. The thickness of shotcrete as the invert should be determined individually, referring to that in the top heading and bench. Although the thickness of shotcrete as the invert can be included in the thickness of invert concrete. The thickness of cast-in-place concrete in the invert should not be lower than the thickness of the lining in the arch and side wall.
   iii) Even if rocks are DI class, the invert can be omitted in case the long-term bearing capacity of the rocks is expected to be sufficient as hard rocks in the bench, and squeezing due to lateral pressure is not anticipated.
4) On wire fabric
   i) For DI rocks, wire fabric is generally placed in the top heading. For DII rocks, wire fabric is usually placed in both the top heading and bench.
   ii) The placement of wire fabric can be omitted when steel fibre reinforced shotcrete (SFRC) is adopted.
5) On the allowable deformation
   For DII rocks, the allowable deformation usually provided in the design is about 10 cm. The a1lowable deformation is provided in the top heading for the bench cut method, whereas it is provided in both top heading and bench for full-face method with auxiliary bench cut. The allowable deformation criteria should be modified from time to time based on the monitoring results during tunnelling works.
6) Even in the case where the design of ordinary section tunnels is applicable, the application of the tunnel support system for large diameter tunnels should be considered when the flattened cross-section such as three centred arches is adopted in the boundary region to large section.

Table 2-8: Standard Tunnel Support System for Road Tunnels (Large Diameter Tunnels)

(Large section tunnels, inner diameter: about 12.5 m to 14.0 m)

Rock mass Classification	1 Cycle Driving Progress (m)	Rock Bolt				Steel Ribs			Thickness of Shotcrete (cm)	Thickness of lining		Allowable Deformation (cm)	Excavation Method
		Length	Spacing		Installation Direction	Top Heading	Bench	Spacing (m)		Side Wall (cm)	Invert (cm)		
			Perpendicular to tunnel Direction (m)	Longitudinal Direction (m)									
B	2.0	4.0	1.5	2.0	Top Heading	-	-	-	10	40	-	0	Full Face method with mini bench cut or short bench cut
CI	1.5	4.0	1.2	1.5	Top Heading, Bench	-	-	-	15	40	(45)	0	
CII	1.2	4.0	1.2	1.2	Top heading, Bench	H-150	-	1.2	15	40	(45)	0	
DI	1.0	6.0	1.0	1.0	Top Heading, Bench	H-150	H-150	1.0	20	40	50	0	
DII	≤ 1.0	6.0	1.0	≤ 1.0	Top heading, bench	H-150	H-200	≤ 1.0	25	40	50	10	

Table 2-9: Standard Tunnel Support System for Expressway Tunnels (Large Diameter Tunnels)

Three-Lane expressway tunnels (Excavation cross-section: 150 m^2 to 200 m^2)

Rock mass Classification	1 Cycle Driving Progress (m)	Rock Bolt				Steel Ribs		Thickness of Shotcrete (cm)	Thickness of lining		Allowable Deformation (cm)	Excavation Method
		Length	Spacing		Yield strength (kN)	Top Heading	Bench		Side Wall (cm)	Invert (cm)		
			Perpendicular to tunnel Direction (m)	Longitudinal Direction (m)								
B	2.0	Top Heading 4.0 Bench 4.0	2.0 [3.8]	2.0	Top Heading 170 ° Bench 170°	--	--	10 (Fibre reinforced in top heading)	40	-	0	Bench cut
CI	1.5	Top Heading 6.0 Bench 4.0	2.0 [2.9]	1.5	Top Heading 290 ° Bench 170°	HH-154	--	15	40	55	0	
CII	1.2	Top Heading6.0 Bench 4.0	1.6 [1.8]	1.2	Top Heading 290 ° Bench 170°	HH-154	HH-154	15	40	55	0	
DI	1.0	Top Heading 6.0 Bench 6.0	1.5 [1.4]	1.0	Top Heading 290 ° Bench 170°	HH-154	HH-154	20	50	70	0	
DII	Specification shall be designed individually											
DIII	Design of portal zones shall be applied											

1) When the advancing drift is applied, the round length is changed from that in this table
2) For class B rocks, high-early-strength mortar is adopted as the bonding material for rock bolts.
3) For CI, CII, DI rocks, although the yield strength of rock bolts is designed as 290kN in the top heading, the yield strength can be changed to 170kN when no problem is anticipated based on the monitoring results. The twist bolt should be adopted when the displacement is large, and an increase in the number of rock bolts should be considered when the strength is still insufficient.
4) High-strength steel ribs are applied for steel ribs.
5) Single reinforcement is generally adopted for the lining of the DI rocks.

Table 2-10: Standard Tunnel Support System for Road Tunnel Portals (Medium Diameter Tunnels)

(Ordinary section tunnels, Inner diameter: about 8.5m to 12.5m)

Excavation Method		1 Cycle Driving Progress (m)	Rock Bolt			Steel Ribs			Thickness of Shotcrete (cm)	Thickness of lining	
			Length	Spacing		Top Heading	Bench	Spacing (m)		Side Wall (cm)	Invert (cm)
				Perpendicular to tunnel Direction (m)	Longitudinal Direction (m)						
Bench cut method, Full face method with auxiliary bench cut.		1.0	4.0 (3.0)	1.2 (0.6)	1.0 (1.0)	H-200	1.0	1.0	25	35	50
Side Drift advancing	Main Tunnel	1.0	4.0 (3.0)	1.2 (0.6)	1.0 (1.0)	H-200	-	≦ 1.0	25	35	≧ 50
	Drift	1.0	2.0 (2.0)	1.2 (0.6)	1.0 (1.0)	H-125		1.0	10	-	-

1) The values in parentheses represent the specification for fore poling.
2) Rock bolts are arranged around the side wall, and the area of arrangement should be extended to the arch if necessary. The standard length of rock bolts is 4 m.
3) Forepoling is implemented in the range of 120 degrees of the crown if necessary to stabilize the crown of the face. The material and method for fore poling should be properly determined so that they will be suited to the site conditions.
4) As a general rale, wire fabric is installed in both top heading and bench for the bench cut method and full-face method with auxiliary bench cut, whereas it is installed in the top-heading for side drift advancing method. This rule does not apply when steel fibre reinforced shotcrete (SFRC) is adopted.

Table 2-11: Standard Tunnel Support System for Road Tunnel Portals (Large diameter Tunnels)

(Large section tunnels, inner diameter: about 12.5 m to 14.0 m)

Excavation Method		Cycle Driving Progress (m)	Rock Bolt			Steel Ribs			Thickness of Shotcrete (cm)	Thickness of lining	
			Length	Spacing		Top Heading	Bench	Spacing (m)		Side Wall (cm)	Invert (cm)
				Perpendicular to tunnel Direction (m)	Longitudinal Direction (m)						
Bench cut Method		1.0	6.0 (3.0)	1.0 (0.5)	1.0 (1.0)	H-200	H-200	1.0	25	25	50
Centre Diaphragm	Main tunnel	1.0	6.0 (3.0)	1.0 (0.5)	1.0 (1.0)	H-200	H-200	1.0	25	45	50
	Centre diaphragm	1.0	3.0 (3.0)	1.2 (0.6)	1.0 (1.0)	H-150	--	1.0	15	--	-
Side Drift Advancing	Main tunnel	1.0	6.0 (3.0)	1.0 (0.5)	≤ 1.0 (≤ 1.0)	H-200	--	≤ 1.0	25	45	≥ 50
	Drift	1.0	2.0 (2.0)	1.0 (0.6)	1.0 (1.0)	H-125		1.0	10	--	≥ 50
Centre Drift Advancing	Main tunnel	1.0	6.0 (3.0)	1.0 (0.5)	≤ 1.0 (≤ 1.0)	H-200	H-200	≤ 1.0	25	45	
	Drift	1.0	2.0 (2.0)	1.0 (0.6)	1.0 (1.0)	H-125	H-125	1.0	10	--	

1) The values in parentheses represent the specification for fore poling.
2) Rock bolts are arranged around the side wall, and the area of arrangement should be extended to the arch if necessary. The standard length of rock bolts is 6 m.
3) The fibre-reinforced plastic bolt (FRP) can be adopted in the center diaphragm of the center diaphragm method and in the drift of center drift advancing method so that the rock bolt can be easily removed and cut when excavating the main tunnel.
4) Forepoling is implemented in the range of 120 degrees of the crown if necessary to stabilize the crown of the tunnel face. The material and method for fore poling should be properly determined so that they will be suited to the site conditions.
5) The foot of the shotcrete shall be supported by the invert to achieve the effects of closure with tunnel supports.
6) As a general rule, wire fabric is installed in both top heading and bench for the bench cut method, top heading center diaphragm method, and center drift advancing method, whereas it is installed in the top heading for side drift advancing method. This rule does not apply when steel fibre reinforced shotcrete (SFRC) is adopted.
7) An accurate examination of countermeasures for unsymmetrical ground pressure in portals is necessary when the size of the tunnel cross-section increases.
8) When the wall type portal is adopted, the thickness of the lining at the joint with the wall should be determined taking into full consideration the difference in the thickness between the wall and lining.

(2) Change of Supports

From a viewpoint of safety and cost-effectiveness, it is important to evaluate visual inspection and monitoring results and to make use of these evaluations in changing supports

When changing supports, available results of visual inspection and monitoring should be correctly evaluated so that once changed the supports will be suited to actual site conditions. In addition to the change of tunnel support systems, tunnel shape and reinforcement works are modified if necessary. If the geology ahead of the tunnel face is investigated by advanced boring etc., or the geology is investigated with precision, results of such investigation should also be evaluated in the process of changing support. Accordingly, if the supports in the original design need to be modified, changes should be made without delay. There are two cases in changing supports: 1) change of supports in unexcavated areas, and 2) change of supports in excavated areas.

1) Change of tunnel support systems in unexcavated areas
The initial tunnel support system is determined on the basis of geological investigation results is just a standard pattern. It is therefore necessary to convert it into a more rational pattern better suited to the actual rock conditions, referring to visual inspection and monitoring results in excavated areas. Changing supports include upgrade and downgrade. The degree of change depends on the difference in rock conditions and displacement from the original design. Accordingly, change of support ranges from relatively minor changes in quantity and materials of support members to the entire modification of tunnel support system by reviewing rock mass classification (refer to Feedback to Design, Construction, and Maintenance). When planning to change supports, it is also necessary to examine reinforcement works to ensure safety and efficiency of work. It may be effective to examine and prepare, in advance, the detailed tunnel support system established by the subdivision of standard tunnel support systems.

2) Change of support in the excavated area
If displacement after excavation is more significant than initially anticipated, or displacement does not converge, or convergence and/or surface settlement would exceed the control criteria, supports may be changed, such as additional rock bolts, additional shotcrete, and placing the invert in order to strengthen the supports already in place. In these cases, to avoid excessive deformation of tunnel support system, shape, materials, and arrangement of additional supports and reinforcement works are studied and implemented without delay.
When measures are required to meet the specified limit of surface settlement or the restriction concerning structures in the vicinity, it is important to examine not only strengthening of the tunnel with additional supports or reinforcement works, but also necessary protection such as cut-off walls in order to provide suitable solutions.

Table 2 12 shows typical changes of support during the tunnel drivage stage.

Table 2-12: Typical Changes of Tunnel Support System during Tunnel Drivage Stage

Items	Typical Changes
i) Shotcrete	- Change of thickness, materials, etc.
ii) Rock Bolts	- Change of length, quantity, strength, bonding material, bonding type
iii) Steel ribs	- Necessity of steel ribs. - Change of size, spacing, materials, etc.
iv) Invert Concrete	- Adoption or rejection of invert - Change of shape, the timing of placing, materials, etc.
v) Other factors	- Change of round length. - Application of reinforcement works. - Examination of lining structure.

2.3.2 Shotcrete

(1) General

Shotcrete is a support material, concrete placed to adhere closely to the surface of the tunnel wall. Since the shotcrete process can be performed immediately after boring and is not affected by the size or shape of the tunnel section, it is the most widely used support material.

The support function of shotcrete is thought to be its resistance to the deformation of rocks which accompanies boring, and the compaction and shear resulting from external forces. These functions of shotcrete provide effective support in various combinations and stabilize rocks. Various concepts of these functions and effects as tunnel support can be considered, as given in Table 2-13, together with conceptual diagrams.

It is necessary to decide specifications for the thickness, strength, etc. of shotcrete, taking into consideration rock conditions, the purpose of the tunnel, etc., to ensure the stability of the tunnel after boring. Further, shotcrete is generally used with other support materials like rock bolts and steel tunnel supports, and in these cases, the use of shotcrete must be planned to effectively maximize the supporting function considering the various possible support functions.

Moreover, in deciding the mix and method of application of shotcrete, attention should be paid to the mechanical function of; 1) whether suitably necessary strength can be achieved immediately the following shotcrete and over the long term; 2) whether sufficient adhesion to the rocks can be achieved; and to the application characteristics of; 3) whether a rebound is contained; 4) whether dust production is contained, etc.

Since the support effect of shotcrete varies with rock conditions, it is necessary to consider its suitability in relation to geology, as explained below.

1) Medium hard, hard rocks
   For medium-hard and hard rocks, where the intervals between the bedding, joints, and other surfaces of discontinuity are comparatively large, and surfaces of discontinuity will control tunnel behaviour, shotcrete is mainly used to prevent the collapse of localized masses of rock, reinforce weak strata, and as a protective covering.

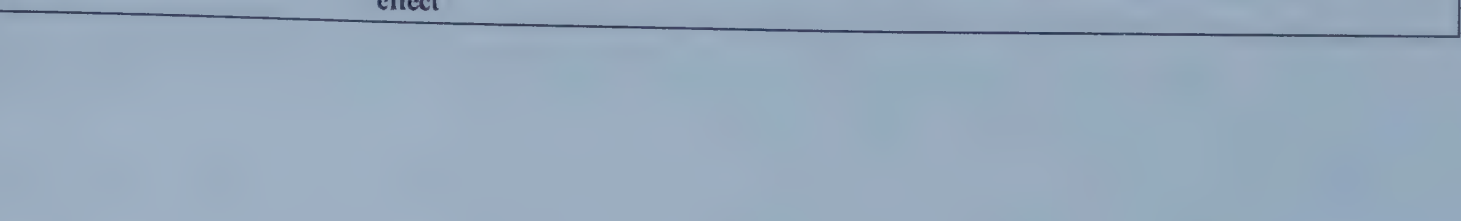
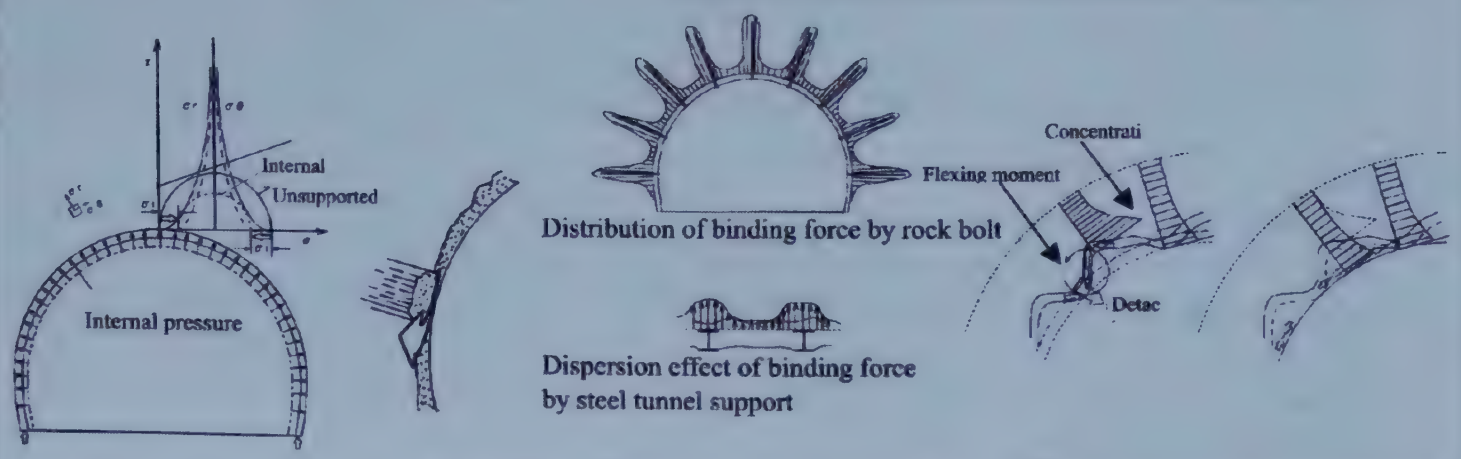
2) Soft rocks
   For soft rocks, where the rocks surrounding the tunnel shows macroscopically continuous behaviour, but rocks have insufficient strength, the stability of the tunnel face is inferior, or plastic earth pressure is increased, shotcrete is mainly used to prevent localized rock falls to provide internal pressure, to level out stress distribution, and as a protective covering against rocks degradation such as slaking.

3) Soil
   For soil, where rocks strength is low and the stability of the tunnel face is inferior, shotcrete is used as the initial protective covering of the cutting face to prevent degradation of rocks, to prevent rock falls, to provide internal pressure, and to level out stress distribution against other support material being used, etc. Further, attention should be paid to the fact that adhesion between shotcrete and rock may be hard to achieve.

4) Fault fracture zones, squeezing rocks
   In rocks where there is large earth pressure and deformation, such as faults fracture zones and expansive rocks, shotcrete is generally used for 3) Soil as mentioned above. However, thickness and material strength are often increased to reinforce compaction resistance, flexing resistance, and shear resistance, and to provide internal pressure.
   Further, in rocks where there is little water inflow and comparatively few fissures, on-site casting of lining concrete may be omitted, and the tunnel finished with shotcrete. When shotcrete is used in this case not only the purpose of the tunnel,

rock conditions, conditions of adjacent facilities, etc., but also strength as a mechanical characteristic of shotcrete, and where necessary compactness and durability must be considered. It is also necessary to respond to conditions with the appropriate mix and application method.

Table 2-13: Function and Effect of Shotcrete

Category of function, effect			Summary of function and expected effect
Function	I	The compression strength of concrete	To resist earth pressure and tangential stress on tunnel inner surface using compression strength of concrete.
	II	Shear strength of concrete	To resist shear stress and shear displacement working on shotcrete by means of the shear resistance and rigidity of concrete. Cohesive strength between rocks and shotcrete is important to keep the resistance action of shotcrete. If the cohesive strength is lost, shear resistance and rigidity of concrete are lost.
	III	Bending strength of concrete	To resist bending stress on shotcrete by means of the bending strength and rigidity of concrete.
	IV	Shear resistance and cohesion between shotcrete and rocks	To hold and disperse the tangential stress, the shear stress and the bending stress from the earth by means of shear strength and cohesion between shotcrete and rocks
	 I axial compaction resistance II shear resistance III flex resistance IV shear resistance, adhesion resistance of concrete – bedrock interface		
Effect	1	Rock fall prevention, retaining of small rock masses	To prevent tunnel wall collapse and rock falls and to secure the work safety.
	2	Application of pressure to rocks from inside the tunnel	To increase the strength of soft rocks around the tunnel by applying outward stress from inside of the tunnel using shotcrete.
	3	Reinforcement of weak rocks and protection of tunnel shape	Shotcrete concrete which penetrates into open cracks in friable rocks around the tunnel can act as reinforcing filler and glue.
	4	Levelling out of stress distribution	To level out tangential stress of earth pressure by means of levelling the tunnel inner surface. To disperse various stresses around the tunnel include tension stress from rock bolts and compressive stress from steel ribs.
	5	A protective covering, prevention of rock weathering	To protect the rock from air and inflow water to prevent rock weathering, softening of rocks, and soil inflow.
	 Internal Unsupported Internal pressure Distribution of binding force by rock bolt Dispersion effect of binding force by steel tunnel support Flexing moment Concentrati Detac ② internal pressure effect ③ weak strata reinforcement effect ④level out effect of radial stress distribution in circumferential stress ⑤level out effect of circumferential stress distribution in radial direction stress		

(2) Mechanical Characteristics of Shotcrete

In general, comparatively high compaction strength is commonly required in the initial stage after application for shotcrete used in tunnels. Strength settings typically required one day after application are as follows; for a two-lane road tunnel, the strength of 5 N/mm2; for a bullet train tunnel, 8 N/mm2; and for a large diameter road tunnel, 10 N/mm2. Further, material age is broadly divided into three stages: the initial stage (one day after application), the early stage (seven days after application), and the longterm (more than 28 days after application).

Design standard strength is generally defined as uniaxial compaction strength 28 days after application, i.e. long-term strength, set at 18 N/mm2 for two-lane road tunnels and a railway tunnel. A higher strength of 36 N/mm2 may be set4) for large diameter road tunnels, to reduce the thickness of spraying, and to increase resistance with the expansive rocks.

From the initial stage to the early stage, it is necessary to make adjustments to unit cement content, water/cement ratio (water/bonding material ratio), and admixture (additive agent), and type of accelerating agent and dosage to ensure required strength. Further, it is necessary to bear in mind that excessively increasing accelerating agent dosage to increase strength in the initial and early stages may depending on the type of accelerating agent, inhibit the promotion of long-term strength.

Further, fibre-reinforced shotcrete, where fibre is added to the shotcrete, may be used for large diameter road tunnels, expansive rocks, or junctions and branches, to improve flexural strength and to prevent the peeling of shotcrete after the development of cracks. In this case, it is best to specify appropriate design indices which can express necessary mechanical characteristics such as flexural strength to meet the purposes of fibre reinforcement. Figure 2 2 shows a provisional example of flexural strength.

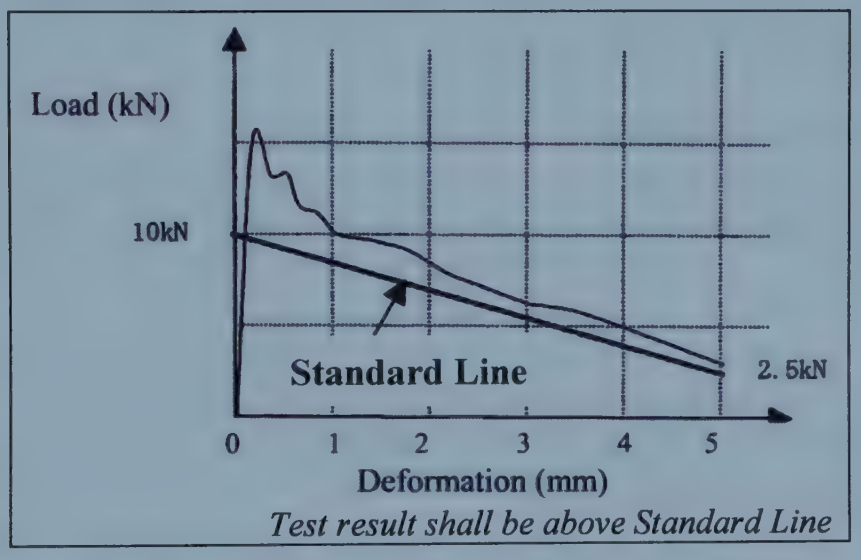


Figure 2-2: Example of Specification of Flexural Strength in Steel Fibre Reinforced Shotcrete

(3) Mix of Shotcrete

There are two types of shotcrete, wet and dry methods, and the final quality of shotcrete by each method is easily affected by the application process. Work performance, as well as the quality of shotcrete, are taken into account in the design of the shotcrete method.

It is necessary that the shotcrete quality be high enough so that initial stage strength is sufficient to adhere to the rocks when applied immediately after boring. Strength in the early stage should be sufficient to ensure the stability of the contiguous portion to the tunnel face during shotcrete working. Finally, long-term strength should be sufficient to support the rocks over the long term, and the shotcrete should possess the necessary durability to maintain the function of the structure. It is vital to keep the volume of dust

produced at the portal at the absolute minimum for the safety of the work environment. Therefore, the main items to consider in shotcrete mix are as follows;

i. strength (initial stage strength, early-stage strength, long term strength)
ii. adhesion
iii. compactness (adsorbed moisture, void volume, permeability, etc.)
iv. durability (freeze/thaw resistance, chemical resistance, carbonation resistance)
v. rebound ratio
vi. the volume of dust produced

There is a difference between materials mixed into concrete and materials adhering to the concrete, but in planning the mix of shotcrete, the former is considered, in basically the same method as poured concrete.

In planning the mix, since the strength of shotcrete depends on spraying method, water inflow conditions of the sprayed face, portal air temperature, humidity, etc., water/cement ratio or water/bonding material info, fine aggregate ratio, maximum dimensions of coarse aggregate, unit cement content, type of admixture, accelerating agent, unit water volume, and aggregate and other materials used, it is essential that the characteristics of these materials must be considered and ingredients selected appropriately. Examples of specified mixes used in a road tunnel and a railway tunnel are shown in Table 2-14.

1) Water/cement ratio, unit cement content]

The water/cement ratio depends on the spraying method. However it is generally in the range of 50 to 65% for the wet method, 45 to 55% for the dry method, and a low 40 to 50% (water/bonding material ratio of about 30 to 40%) for high strength shotcrete. Unit cement content is high compared to normal poured concrete, based on application experience to date, to overcome work efficiency, to ensure initial stage strength and adhesion, to reduce rebound ratio and volume of dust produced, and to improve the ability to be pumped.

2) Fine aggregate ratio, aggregate

The fine aggregate ratio is greatly affected by work efficiency such as the ability to be pumped, and the appropriate value varies with the diameter and grading of aggregate, particles, and application method. It is generally about 55 to 70%.

It is important to use the appropriate grade of aggregate to achieve strong compact shotcrete. Fine aggregate composes proportionally the largest volume of materials in shotcrete and has a large impact on work efficiency such as the ability to be pumped. As a result, it is necessary to pay attention to the fact that it has a large effect on quality and rebound ratio.

In the case of shotcrete, as particle diameter becomes larger, rebound ratio increases, and the ability to be pumped decreases. The maximum dimensions of coarse aggregate used are normally 10 to 15mm.

3) High-performance wetting agents and other additive agents

High-performance wetting agent has high water reducing effect, reduces water volume, slump, retains workability, improves durability and adhesion, and reduces rebound. Examples of it being used in shotcrete using low dust accelerating agent, high strength shotcrete, and fibre reinforced shotcrete are increasing.

As for dosage increases, viscosity becomes excessive, and work efficiency may become worse.

4) Accelerating agent

An accelerating agent is an additive agent used to promote the strength of concrete in the period from a few hours after application to about one day after application. It is important to use a product appropriate to the rock conditions of the tunnel.

Since dosage varies with spraying method, spraying mix, spraying position, water inflow conditions, etc., it is necessary to confirm it in advance.
Note that if the dosage is excessive, prescribed long-term strength may fail to develop.
The main types used currently are as follows:

i. Quick setting cement mineral type (calcium aluminate type): Uses the same raw materials as cement, promotes hydration of cement, and exhibits self-hardening of accelerating agent itself. There is a tendency for long-term strength and durability to be lower than that for concrete without additives. The majority of accelerating agents used in Japan are of this type.
ii. Ultra quick cement mineral type (calcium sulfoaluminate type): Promotes hydration of cement and exhibits is remarkably low, but the dosage is high at about 10%. Since dosage has little effect on long-term strength, this type is used in high-strength shotcrete.
iii. Alkali free liquid accelerating agent, low alkali liquid accelerating agent: Weak alkaline or neutral to weak acid liquid accelerating agent, which can reduce the barrier caused by high alkali content in the concrete. Since these accelerating agents are not self-hardening, in contrast to (1) and (2) above, there is a tendency for hardening times after spraying to be longer.

5) Silica fume, pulverized limestone admixture

Silica fume is spherical microgranules mainly comprised of amorphous silicon dioxide (SiO_2) formed during the production of ferrosilicon and silicon metals. Silica fume is added in the place of cement or aggregate, generally in a proportion of about 5 to 10% of cement by volume. By using it with high-performance wetting agent, improvements in long-term strength can be expected.
Limestone (calcium carbonate) is limestone pulverized to a specific surface area of about 3000 to 7000 cm^2/g. In bullet train tunnels, limestone microgranules of diameter less than or equal to 0.15mm are used as a partial substitute for fine aggregate, in a proportion of about 15% of fine aggregate by weight, to suppress rebound and dust production.
In addition to these additives, blast furnace slag, fly ash, coal ash, etc. may be substituted for added binder, cement, or aggregate.

6) Dust reduction agent

The main component of a dust reduction agent is a high molecular compound such as cellulose type or acrylamide type, suppressing dust production by coagulation effect of thickening effect. In wet method application, it is normally added to the concrete at the bather plant. In dry method application, it may be added during the mixing of the concrete or dissolved in water added to the concrete. For wet method application, to avoid an uneven mix, it is necessary to confirm the mixing time in a trial mix. Further, the reduction of dust volume produced at the source is most effective as a measure against dust in shotcrete. In relation to the mix, in addition to a dust reduction agent, low dust accelerating agent, or thickening pulverized materials such as silica fume are used. However, if the dosage is large, the viscosity becomes excessive, and work efficiency may worsen.

Table 2-14: Examples of Mix

(a) Road tunnel

		Maximum dimension of coarse aggregate Gmax (mm)	Slump (cm)	Cement C (kg)	Accelerating agent Cx%
Road	Normal mix dry method	15	-	360	5.5
	Normal mix wet method	15	8	360	5.5
Expressway	High strength mix	10	18 ±2	450	10.0
	Standard mix	15	8 ±2	360	7.0

(b) Railway tunnel

	Maximum dimensions of coarse aggregate (mm)	Slump *1 (cm)	Fine aggregate ratio (%)	Water/ bonding material ratio*2 (%)	Cement (Kg)	Accelerating agent (C+SF) x %	High performance wetting agent	Pulverized limestone	Silica fume SF	Notes
High quality spray	10-15	8 ±2 14 ±2	60-65	55-60	342	4-7	As necess ary	About gx15%	18 Kg	Conveyed by pump, SEC mix.

**1 In the case of conveyance by pump, the slump is about 8±2 cm, In the case of conveyance by air, 14±2 cm.*
**2 W/(C+SF)*

(4) Design Thickness of Shotcrete

It is necessary to decide and appropriate thickness for the design thickness of shotcrete, taking into account rock conditions, size of tunnel diameter, work efficiency, etc.

Where effective earth pressure and deformation are large, as in the case of expansive rocks; in which load-bearing ability and ductility is small, as in the case of unconsolidated rocks; or where the overburden is sparse and it is necessary to minimize the impact on the surroundings, it is necessary to spray comparatively thickly, mainly to anticipate the axial compaction resistance of shotcrete, to strengthen the resistance and rigidity of tunnel support. However, in hard rock where earth pressure has almost no effect, to prevent rock falls, where the shear resistance of shotcrete is the main characteristic of shotcrete expected to be encountered, the design thickness of shotcrete may be comparatively thin.

Normally, the design thickness of shotcrete follows a standard tunnel support system based on past application experience. However, where the standard tunnel support system cannot be applied due to special design conditions, it is best to determine design thickness based on analytical techniques, paying sufficient attention to the stability of rocks surrounding the tunnel, allowable displacement, stress conditions of support material, etc.

Further, where biased pressure or a large flexing moment is predicted, it can be prudent to increase the thickness sprayed. However, where thickness sprayed will be greatly increased, reducing the thickness sprayed by using high-strength shotcrete can be effective.

There are two ways of approaching to design thickness of shotcrete: minimal thickness sprayed and average thickness sprayed. The design thickness sprayed over the whole circumference of the tunnel must be greater than or equal to the minimal thickness sprayed. In the case of average thickness sprayed, it does not matter that some places have less than the design thickness sprayed on them, so long as the average thickness over the circumference is greater than or equal to the design thickness. In the case of blast boring on medium hard rocks or hard rocks, the boring face will not always be leveled, and if excessive efforts are made to remove partial protrusions, the slack of the rocks and the volume of the overbreak may increase considerably and not be cost-

effective. Therefore, for these rock conditions only, partial protrusions of rocks into the shotcrete may be permitted.

(5) Reinforced Shotcrete

Where reinforcement is necessary for shotcrete, the strength characteristics of shotcrete materials may be improved, or the thickness sprayed may be increased. Of these two methods, the strength characteristics of shotcrete materials can be improved by reinforcement with wire nets or fibres to improve shear strength, tensile strength, and toughness, in addition to increasing compaction strength.

1) Wire netting

Wire netting is used to improve the shear reinforcement of shotcrete, to prevent collapse for a long time, and to improve toughness after the appearance of cracks.
Generally, wire netting is used in a variety of rock conditions from soil to hard rocks. However, depending on rocks on rock conditions, the main purpose of the reinforcement may differ. In rocks which show a large degree of deformation, such as expansive rocks, many cracks may form in shotcrete, and lumps of concrete may peel. In this case, it is common to use wire netting to prevent peeling and to improve toughness. Where there are many joints and fissures in hard rock, with concern that rock masses might collapse suddenly, wire netting is commonly used to reinforce the shear strength and to improve the toughness of shotcrete. Further, in the soil where adhesion between shotcrete and rocks is poor, wire netting can be expected to prevent the peeling of shotcrete during application.
Welded wire netting is generally used, commonly with a mesh of 100 x 100mm or 150 x 150mm, and a wire gauge of about 3.2 to 6.0mm.

2) Fibre-reinforcement

Fibre-reinforced shotcrete may be effectively used in places where deformation is large and toughness is necessary, or in places where the stress distribution of tunnel support is complex, and toughness and flex resistance is necessary. Examples of these places include expansive rocks; portals, which are easily affected by vertical load or biased stress, as overburden is scant and the earth arch is difficult to form; and junctions and branches, where tunnel diameter is large and shape is complex compared to the main tunnel.
In comparison to regular shotcrete, the indices of peak strength, which are tensile strength and flex strength, are increased only slightly, but the residual strength after peak strength is high, and the marked improvement in toughness and ductility after the appearance of cracks is the most important characteristics of fibre-reinforced shotcrete. In addition, shear strength, impact resistance, and freeze/thaw resistance is improved. The fibre-reinforced shotcrete has a high resistance to cracks, avulsion fractures and flaking. Since it is more difficult for cracks to penetrate into fibre-reinforced shotcrete and waterproofing is easier to maintain, improving the durability of the structure, fibre-reinforced shotcrete may be used in tunnels finished with shotcrete. Where the thickness of shotcrete is restricted in the reconstruction or repair of existing tunnels, it is an effective material in constructing thin-walled lining.
The proposed strength of fibre-reinforced shotcrete, as well as general shotcrete, specifies flexural strength.
Steel fibre, and non-steel fibre are used for fibre-reinforced shotcrete. The fibre used for reinforcement of shotcrete commonly has a length of about 25 to 35mm, an aspect ratio (length/diameter) of about 40 to 60, and the volume fraction of that fibre is commonly about 0.5 to 1.0% by volume, taking into consideration the work efficiency. Further, as shown in the examples of mixes for fibrereinforced shotcrete

in Table 2-15, to ensure work efficiency, it is common to increase the volume of cement and use high-performance wetting agent.
The polypropylene fibre, vinylon fibre, etc. are used as non-steel fibre reinforcement. The constraints and strength characteristics when using these non-steel fibres differ from those for steel fibres, and it is necessary to confirm them in advance.

Table 2-15: Examples of Fibre Reinforcement Mixture

(a) Mixture of steel fibre reinforced high strength shotcrete in 1 m^3 (σ 28 = 36 N/mm2)

Cement C	Water W	Fine aggregate G	Coarse aggregate G	Addictive agent	Accelerating G Agent	Steel Fibre
450 kg	202.5 kg	1115 kg	478 kg	1.76 kg	45.0 kg	78.5 kg

(b) Mixture of vinylon fibre reinforced shotcrete in 1 m^3

Fine aggregate Ratio S/a	Cement C	Silica Fume	Accelerating agent	Vinylon fibre	Wetting agent
65 %	360 kg	5 %	5.5 %	10 kg	2 %

2.3.3 Rock Bolts

(1) General

Rock bolts are composite materials comprising a core made out of steel rod placed approximately in the centre of a hole drilled from the tunnel wall surface into the rocks; and anchorage material to integrate the core with the rocks surrounding the hole; and ahead, which is a plate and nut to bind the shotcrete with the core. As opposed to shotcrete and steel tunnel supports, the characteristics of rock bolts are that they exhibit a support function from within the rocks.

It is thought that the support function of rock bolts is mainly to suppress relative displacement parallel to or at right angles to the fissure plane in the case of medium-hard to hard rocks which have developed fissures and to suppress relative displacement in a direction radial to the tunnel between the tunnel wall surface and the rocks in the case of soft rocks and soil. These functions and effects of rock bolts as tunnel supports are shown in Table 2-16. Further, the relationship between rock conditions and the function and effect of rock bolts is shown in Table 2-17.

In planning rock bolts, it is necessary to decide the specification of the anchorage system, anchorage material, distribution and dimensions, material and shape, taking into consideration geologic structure and the relationship to the function and effects of rock bolts, rock conditions, the purpose of the tunnel, and work efficiency, to ensure the stability of the tunnel after boring. Also, where the function of rock bolts is projected for the long term, it is necessary to consider the degradation of resistance through the corrosion of steel materials. Further, rock bolts are generally used with other support materials such as shotcrete and steel tunnel support, and the design of rock bolts must include an evaluation of the total support effect, as well as consideration of the individual support effects.

Since the support effect of rock bolts depends on rock conditions, it is necessary to take into consideration suitability for the geology, as explained below:

1) Medium-hard rocks, hard rocks
   In medium rocks and hard rocks which has developed fissures, rock bolts bind unstable masses of rock partitioned by fissures to deep rocks, and have the effect of inhabiting the peeling and ejection of those masses of rock, i.e. suspension effect and stitch effect can be expected. Meanwhile, rock fragments smaller than the placement interval of rock bolts, which have separated from the rocks, are supported by shotcrete. Shotcrete supports the load by adhesion to the rocks, however, where adhesion between the shotcrete and the rocks is lost, by stitching the shotcrete to the rocks, rock bolts can be expected to support that load.

2) Soft rocks, soil
   In the case of soft rocks or soil with little strength, the axial forces developed by the rock bolt act on the shaft wall through the medium of the shotcrete, demonstrating an apparent internal pressure effect, and the suppression of plasticization of rocks surrounding the tunnel and its extension can be expected. Further, as in the case of medium-hard rocks and hard rocks, depending on the placement of rock bolts, the improvement of shear resistance of rocks, and residual strength after yield, and the apparent effect of improvement to the nature of the rocks can be expected. In soil with shallow overburden, this effect can be expected, and reinforcement in the vicinity of arch legs and fore piling may be carried out.
   In addition to these effects, stratified rocks can be bound by rock bolts, and the formation of a reinforced arch zone surrounding the tunnel boring face can be expected.

Table 2-16: Concepts of Function and Effect of Rock Bolts

Category of Function, effect			Summary of Function and Expected Effect.
Function	I	Tensile resistance of rock bolt	Function to suppress displacement relative to rocks in the axial direction, by the tensile resistance of the rock bolts in that direction.
	II	Shear resistance of rock bolt	Function to suppress displacement relative to rocks at right angles to the axial direction, by the shear resistance of the rock bolt in that direction.
Effect	1) Reinforcement of bedrock	a) Suspension effect	In medium-hard to hard rocks, which has developed fissures, binds unstable masses of rock partitioned by fissures to deep bedrock, and inhibits the peeling and ejection of those rock masses.
		b) Stitch effect	
		c) Effect of improvement to the nature of the bedrock	In medium-hard to hard rock, if rock bolts are placed to intersect with fissures, the shear strength of the fissure plane is increased, and the apparent effect of improvement to the nature of the rocks can be expected. Also, in the case of soft rock or hardly weathered rocks, depending on the placement of rock bolts, the improvement of shear resistance of rocks, and residual strength after yield, and the apparent effect of improvement to the nature of rocks can be expected.
	2)	Internal pressure effect	In the case of soft rock or hardly weathered rocks, the axial forces developed by the rock bolt act on the shaft wall through the medium of the shotcrete, demonstrating an apparent internal pressure, and the suppression of plasticization of bedrock surrounding the tunnel and its extension can be expected.
	3)	Shotcrete support effect.	Rock fragments smaller than the placement interval of rock bolts, which have separated from rocks, are supported by shotcrete. Shotcrete supports the load by adhesion to rocks. However, where adhesion between the shotcrete and rocks is lost, by stitching the shotcrete to rocks, rock bolts can be expected to support that load.

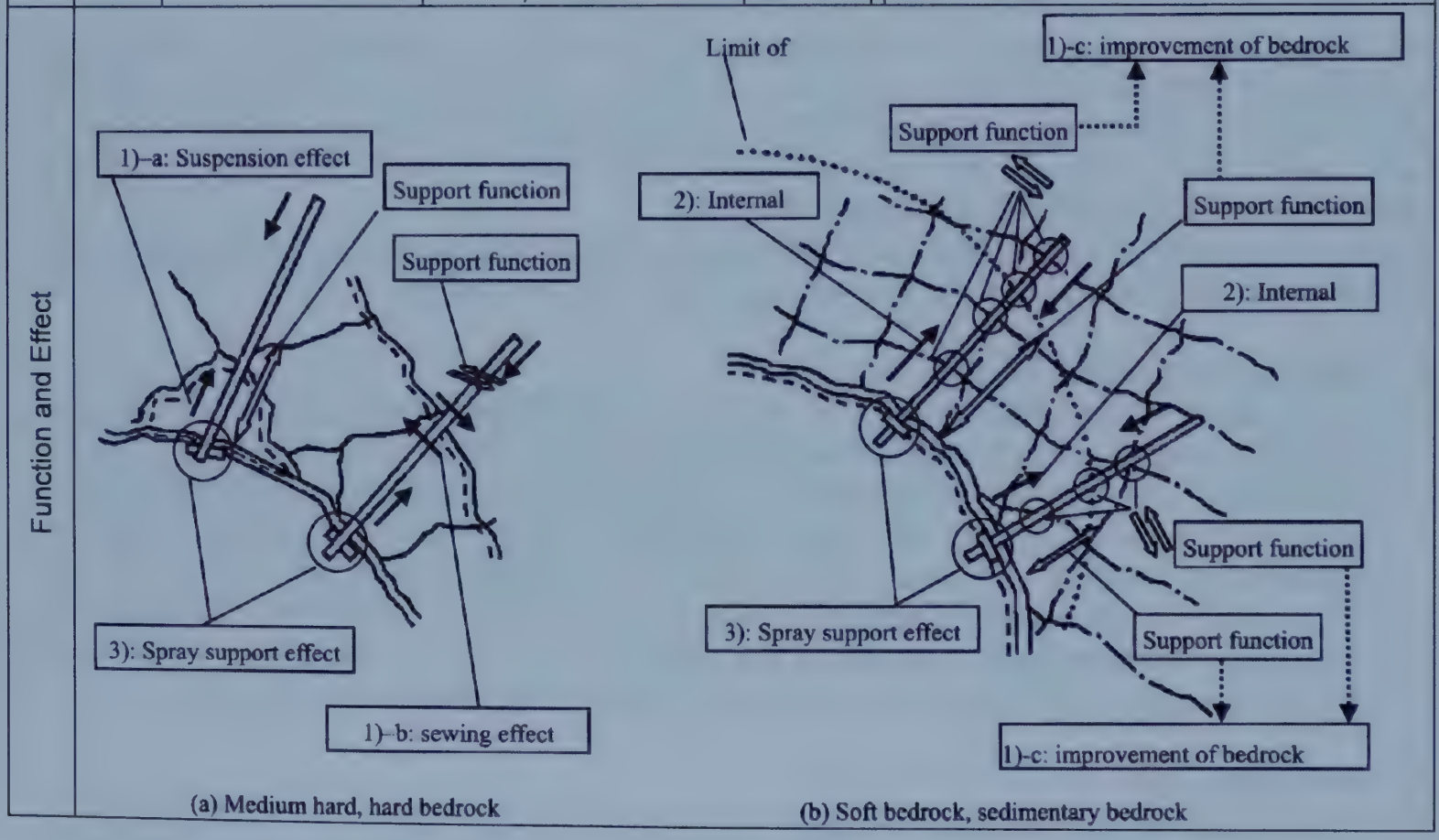


Table 2-17: Relationship between Rock conditions and Function and Effect of Rock Bolts

Rock Condition		Discontinuous ⟷ Continuous
		Hard Rocks ⟷ Soft Rocks Soil
Support Function	Axial Tensile Resistance (perpendicular rigidity and Strength)	Suspension Effect ⟷ Internal Pressure Effect
		← Spray Support Effect →
	Sear Resistance at Right Angle to Tunnel Axis	Stitch Effect ←
		Effect of improvement of Rocks

(2) Rock Bolt Anchorage System

The basic rock bolt anchorage system consists of a complete anchorage system, the anchoring of the whole length of the rock bolt in the rocks. There are two types of complete anchorage systems depending on the rock bolts anchorage method: anchorage material type and friction type. They are categorized as explained in Figure 2 3, based on structure and type of anchorage material. The characteristics and limits of use for both of these anchorage methods are explained in Table 2 18.

The anchorage material type is a method using anchorage material such as rapid hardening mortar to fix the rock bolt in the rocks. This method can be divided into two types: the pre-plugged type, where the rock bolt is inserted and fixed into a hole previously plugged with anchorage material such as rapid hardening mortar; and the post-grouting type, where ultra rapid hardening mortar or resin type anchorage material is injected after the rock bolt is inserted into the hole and positioned. The pre-plugged type has most commonly been used where the soundness of the hole wall is good. The post-grouting type has been used where the rocks are brittle, since the benefits of grout injection will result in an improvement to the rocks in addition to the effect of the rock bolt. Moreover, where the whole wall is sound, but there is the concern that the rapid hardening mortar or the pre-plug will flow out even though prevention measures have been employed against water inflow, a capsule containing mortar or resin and hardening agent may be used. Further, where maintenance of the hole wall while inserting the rock bolt is difficult due to an unconsolidated layer of extremely low strength rocks or talus, it is necessary to consider using a post-grout type self-drilling rock bolt.

The steel pipe expansion type is a representative example of the friction type. In this method, the rock bolts are anchored by using a high-pressure pump to expand a steel pipe rock bolt in the hole. The surface of the rock bolt is pushed directly against the hole wall by the high pressure, and friction resistance against the slip works on the whole bolt-hole wall interface. The friction type is commonly employed where there is the concern that the mortar will flow out even though prevention measures have been taken against water inflow, where there are difficulties ensuring mortar quality, or where there is a desire to exploit the benefits of the rock bolt as soon as possible after placement. Note that caution is necessary for its use in soft rocks and soil, as it may be difficult to achieve the necessary anchorage force.

Rock bolts show their effect through their capacity to resist axial tension and shear at right angles to the axis after being anchored in rocks. Therefore, in planning rock bolts, it is necessary to sufficiently consider the yield load of the rock in relation to tension and shear, and the anchorage force of the anchor, referring to the strength and deformation characteristics of the materials used and prior planning experience. Pulling resistance must be evaluated based on the results of pulling tests carried out to determine whether the anchorage force of the rock bolt is sufficient. Pulling resistance is normally achieved by the anchorage force arising between the rocks and the anchorage material, however, care is necessary as it differs based on rock conditions, anchorage system, anchorage material, distribution and dimensions, material, and shape.

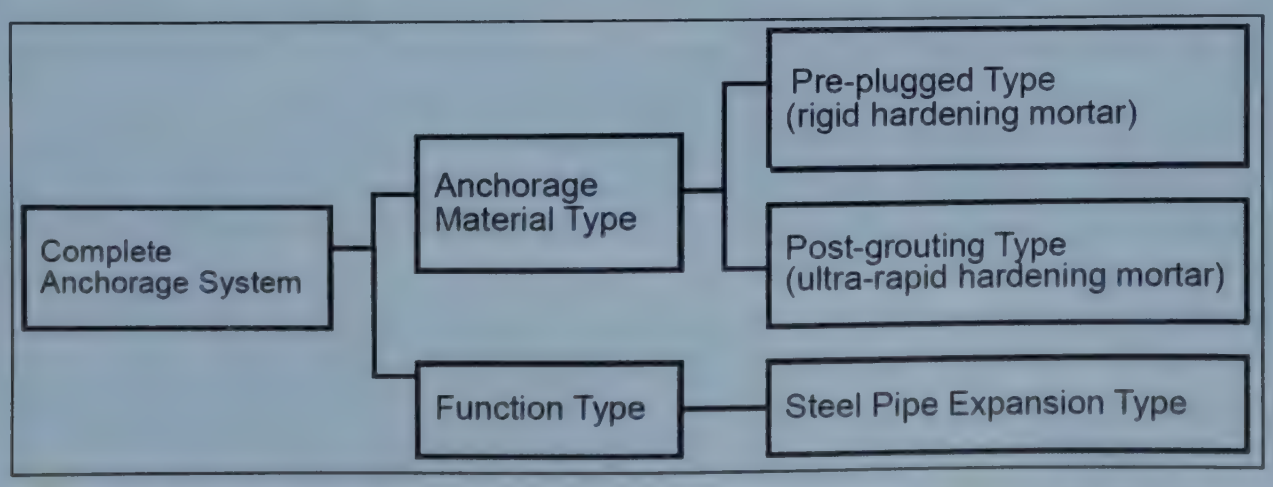
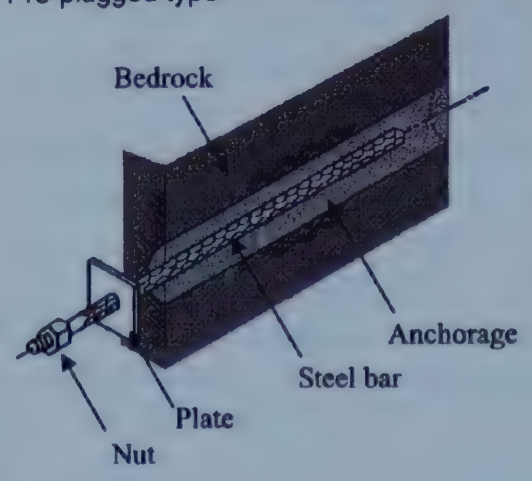
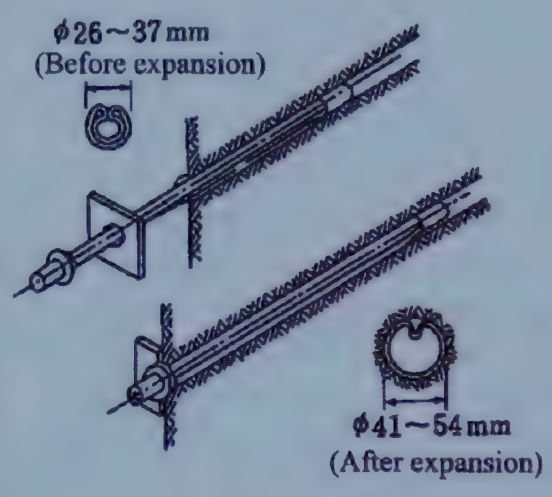


Figure 2-3: Classification of Rock Bolts for Complete Anchorage Systems

Table 2-18: Summary of Complete Anchorage Systems

	Anchorage Method	Characteristics	Application Limit	Schematic drawing
Anchorage Material Type	In the pre-plugged type, the core is inserted and fixed into a hole plugged with anchorage material. In the post-grouting type, anchorage material is injected after the core is inserted and fixed. Rapid hardening mortar is used as anchorage material for pre-plugged type, and ultra-rapid hardening mortar or resin is used as anchorage material for the post-grouting type	Using anchorage material, the whole length of the core is anchored to the bedrock. There are a variety of types, to match bedrock conditions (Fissures, water inflow conditions), soundness of hole wall, etc.	Can be applied to various types of bedrock, not only hard rock, medium-hard rock, soft rock, sedimentary bedrock, but also expansive bedrock.	Pre-plugged type 
Friction Type	Anchorage is achieved by friction by attaching the core close to the surface of the hole wall/ Steel pipe expansion type is representative.	In steel pipe expansion type, a steel pipe with its leasing tips closed off is inserted into a drilled hole. Instantaneous support is achieved by expanding the steel pipe by injecting high pressure. It is necessary to sufficiently consider the reduction of durability such as corrosion of the surface of the steel pipe and degradation of the pushing force added to the surface of the hole wall.	Friction type may be applied to bedrock where water inflow is large. In steel pipe expansion type, since a large degree of plastic deformation in the drilled hole is possible in the radial direction, it can be applied to a wide range of bedrock, so long as the hole wall is sound.	Steel pipe expansion type 

(3) Rock Bolt Anchorage Material

Rock bolts are effective by resisting axial tension and shear at right angles to the axis. Therefore, it is necessary to ensure sufficient anchorage force between the rock bolt, anchorage material, and rocks for effective support.

Anchorage material chosen must achieve sufficient anchorage force along the length of the anchor, taking into consideration rock conditions, work efficiency, etc. Necessary conditions for anchorage material are large anchorage force in the early stage and long term, durability, and good plugging.

It is necessary to consider both adhesions between rock bolt and anchorage material and adhesion between anchorage material and rocks in examining the anchorage force in rock bolts for the anchorage material type. There is usually no problem with adhesion between rock bolts and anchorage material generally used where a sufficient plug is obtained through the normal application. However, it is necessary to exercise care as sufficient adhesion between anchorage material and rocks may fail to be achieved depending on rock conditions. Especially where rocks are of low strength, and anchorage force between the rocks and anchorage material cannot be relied upon, anchorage materials which improve rocks may be used, or the pulling resistance expected from the rock bolt may be decided by calculating back from the achievable anchorage force. Where the pulling resistance expected from the rock bolt and its effect has been revised, it is necessary to consider revising the other support material.

Mortar and resin are used as anchorage materials. Premix type rapid hardening mortar, made at factory by mixing rapid hardening cement, additive agents, and fine sand, which has little variation in quality, is generally used.
Anchorage material may be applied by the pre-lugged type method or the post-grouting type method. Choice of application method is made taking into consideration rock conditions, water inflow conditions, etc.
Further, where long rock bolts are placed in the vicinity of the crown, the plug of the anchorage material may be insufficient; and in unconsolidated rocks like talus, the hole for inserting the rock bolt may collapse. In these cases, it is necessary to sufficiently consider the application method of the anchorage material.
In this way, it is necessary to take into sufficient consideration work efficiency such as the ability to be drilled and plugging of anchorage material; and rock conditions such as soundness of hole wall, the outflow of anchorage material due to water inflow, and leaks from open fissures, in choosing the anchorage system and anchorage material for rock bolts to achieve sufficient anchorage force. Examples of ideas of application limits for anchorage systems and anchorage material for different rock conditions are shown in Table 2-19.

Table 2-19: Concepts of Application Limits for Anchorage Systems and Anchorage Material

Condition of Hole wall	Anchorage system		Anchorage Material	General Rocks				Special Rocks		Special condition					Notes
				Medium Hard rock	Hard rock	Soft rock	Sediment	Unconsolidated	Expansive	Talus	Boulders	Many fissures	Disturbance of drill holes	Water inflow	
Autonomous	Anchorage material type	Pre-plugged type	Rapid Hardening	++	++	++	+	+	+	--	--	+	+	--	Note the ability to insert plug hose, the outflow of the mortar due to water inflow
			Capsule (mortar type, resin type)	++	++	++	--	--	--	--	--	+	--	+	Note the ability to insert a capsule, volume of the plug, outflow due to water inflow.
		Post-grouting type	Mortar type	--	--	--	+	+	++	+	--	+	+	+	Note the ability to insert rock bolt, leak of anchorage material, hardening performance underwater inflow.
			Resin type	--	--	--	+	+	++	+	--	++	+	++	
	Friction Type	Steel pipe expansion type		+	++	+	--	--	--	--	--	--	--	++	Note Friction for poor bedrock.
Not Autonomous	Anchorage material type	Post-grouting type (Self-Drilling)	Mortar type	--	--	--	++	++	++	+	++	+	++	+	Note drill performance, leak of grouting material, hardening performance underwater inflow.
			Resin type	--	--	--	++	++	++	++	++	++	++	++	

++: applicable, +: depending on condition, -: note applicable

(4) Layout and Dimensions of Rock Bolts

1) The layout of rock bolts
The layout of rock bolts is decided taking into consideration rock conditions, size of tunnel diameter, shape, boring method, work efficiency, etc.
Where the purpose is to support the slack area arising from the boring and its load, the location of positioning, the number of rock bolts positioned, and their interval can be decided based on the relationship between the load able to be supported,

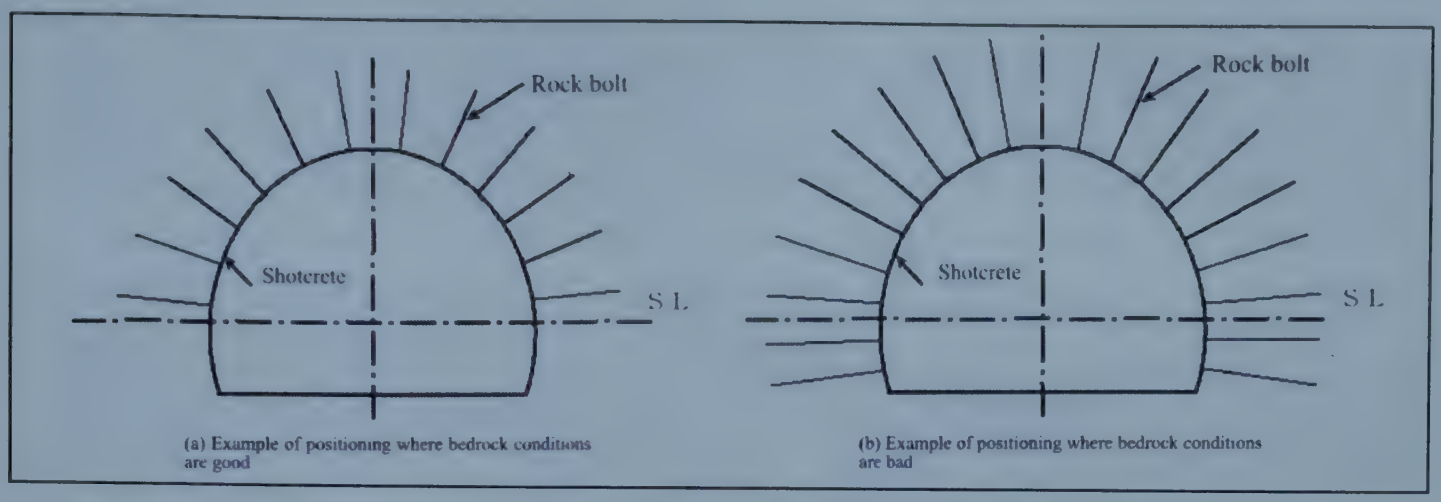
and the load which ought to be supported, by each rock bolt. However, normally layout is decided using a systematic arrangement of bolts, with a tried and tested standard tunnel support system chosen based on the rock conditions. The method of choosing a general standard design of a tunnel support system is usually effective. Where the purpose is to support medium hard rocks or hard rocks which have developed fissures, the number of rock bolts, their interval, and direction can be decided to prevent peeling, hypothesising the potential of mass of rock to peel based on the interval between fissures in the rock, their length, and size. Further, in deciding the layout of rock bolts, it is necessary to take into consideration a positioning interval that will allow adjacent rock bolts to support each other.
Various layouts of rock bolts can be considered, depending upon rock conditions and tunnel diameter. Examples of the layout of rock bolts based on differing rock conditions are shown in Figure 2-4.
As shown in (a) of Figure 2-5, rock bolts are normally positioned as system bolts transecting the tunnel, radiating at right angles to the boring face, to be effective. However, as shown in (b) and (c) of Figure 2-5, there are cases where they are positioned at an angle to the longitudinal direction. In the case of (b), this positioning is adopted where, due to rock conditions, ring cutting is performed in order to stabilize the tunnel face. In the case of (c), this positioning is to reinforce the rocks at the forward part of the tunnel face.
Even where the tunnel diameter is extremely large or extremely small, the concept for layout and length of rock bolts is the same as above. However, for example, in the detailed plan of a road tunnel, the following formula[1)] is referred to, with the stipulation that the application of this formula is limited to the case of system rock bolts suitable for medium-hard to soft rocks.

$$L = \frac{W}{3} \; to \; \frac{W}{5} \; or \; L \geq t \,,$$
$$p = 0.2L \; to \; 0.5L$$

where, L = each length of rock bolt
W = tunnel diameter
t = distance between the last rock bolt and tunnel face
p = positioning interval between rock bolts in the transecting direction

2) Dimension of rock bolts
The length of rock bolts is best decided so as to reinforce the area affected by boring, however, this may change depending on the expected effects.
In places where the stitch effect or the suspension effect is expected from the rock bolts, it is necessary to consider using rock bolts of a length reaching the mass of rock separated by fissures or outside the slack area. In the case of internal pressure or improvement to the rocks, rock bolts of a length reaching beyond the slack area are not always necessary, since the formation of a structure functioning to integrate the rock bolt with the rocks can be expected. However, it is necessary to consider a length able to ensure sufficient anchorage to reduce deformation to the rocks surrounding the tunnel while including the function of the arch structure in the rocks. Meanwhile, the diameter of rock bolts can be decided by the mass of the rock to be supported by each rock bolt and the force of resisting shear in the rocks, however, 21 to 25mm diameter of rock bolts are generally used taking into account the work efficiency.
In rocks with extensive deformation, a large axial force against the rock bolts canaries, causing the rock bolt to fracture within the rocks. It is necessary to consider increasing the number of rock bolts and substituting rock bolts with a core material with a resistance larger than normal. Moreover, where a large axial force

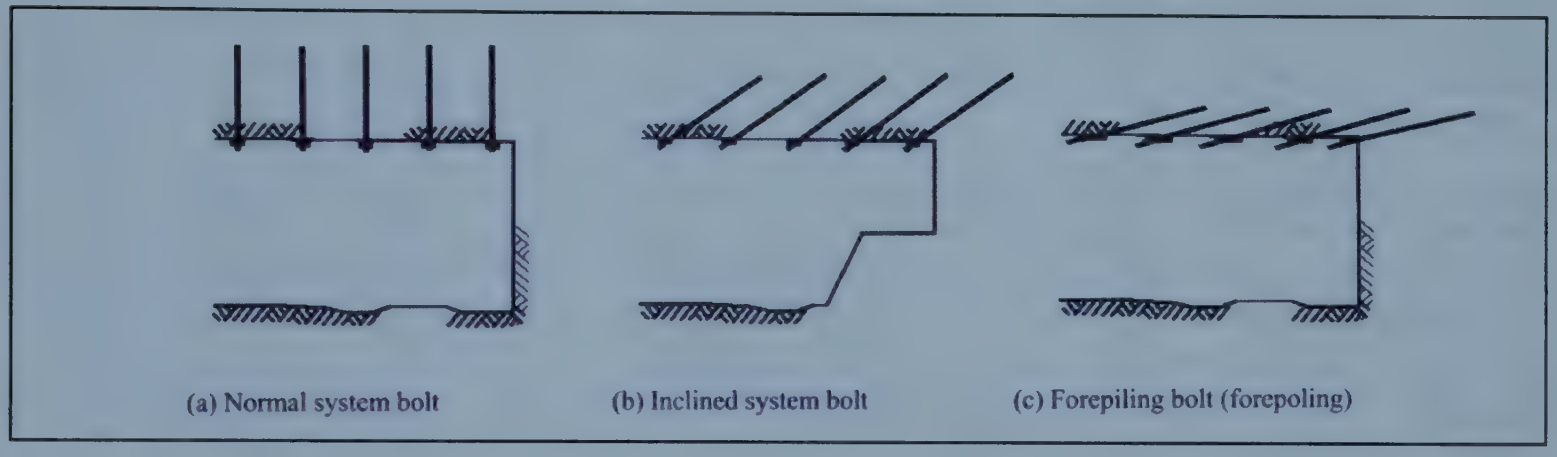
against the rock bolt does not arise in spite of large deformation, there is the possibility that large deformation is occurring deep in the rocks, and the addition of long rock bolts can be considered effective.



(a) Example of positioning where bedrock conditions are good

(b) Example of positioning where bedrock conditions are bad

Figure 2-4: Examples of Layout of Rock Bolts in the Tunnel Transecting Direction



(a) Normal system bolt

(b) Inclined system bolt

(c) Forepiling bolt (forepoling)

Figure 2-5: Examples of Layout of Rock Bolts in the Tunnel Longitudinal Direction

(5) Material and Shape of Rock Bolts

The material in a rock bolt shall have the necessary strength and extensibility characteristics to meet the rock conditions and the purpose of use of that tunnel. For these purposes, extensibility characteristics mean the potential for deformation while retaining a load.

Rock bolts in a complete anchorage system may be torsion bars, deformed bars, threaded deformed bars, or steel pipe expansion type, Table 2-20 shows the mechanical properties of anchorage material type and friction type rock bolts. Further, the resistance of rock bolts is generally expressed as the value at the location with the smallest diameter area, at the threaded portion.

Since rock bolts mainly function as tension members, tensile strength shall be large. However, there are cases where they must have good extensibility characteristics at the same time to prevent a sudden collapse of rocks. Further, since adhesion between the rock bolt and the anchorage material is affected by the shape of the surface of the core material, it is necessary that the shape be appropriate.

Material of SD345 or more is commonly used for the rocks where the large axial force on the bolt is not anticipated. In rocks with large deformation, methods may be employed such as 1) increasing the number of rock bolts; 2) using thicker rock bolts than usual; or 3) using high tensile strength steel. It is necessary to consider the pulling resistance, work efficiency, marketability, and economics of the rock bolts. Where larger resistance is necessary, generally it is more economical to use material of larger diameter without changing the material. However, attention should be paid to the fact that work efficiency deteriorates as the weight of steel materials increases.

Where a borehole is widened and rocks are to be reinforced in advance, in place of a steel bar, a fibre bolt, made of glass fibre or carbon fibre reinforced with strengthened

plastic, may be used, so that it may be easily cut by boring in the later process. In this case, it is necessary to note that although the tensile resistance of the rock bolt is as large as that for steel material, the degree of roughness on the surface of the rock bolt, and the shape.
A straight angle bearing plate of dimension 9 x 150 x 150 is generally used. However, in rocks with extensive deformation, it is necessary to choose the material and shape of the plate, taking into account the mechanical properties of the rock bolt. Further, it is also necessary to choose the nut taking into account the mechanical properties of the rock bolt.

Table 2-20: Mechanical Properties of Anchorage Material Type and Friction Type Rock Bolts

Category	Code	Nominal diameter of the bolt	Mechanical properties of the threaded portion			Mechanical properties of raw material			Unit mass (Kg/m)
			Cross-section area*1 (mm^2)	Yield Load (kN)	Fracture Load (kN)	Cross Section area (mm^2)	Yield Load (kN)	Fracture Load (kN)	
Torsion bar	STD 510*2	TD21	303	154.5	209.1	369	188.2	254.6	2.9
		TD24	353	180.0	243.6	446	227.5	307.7	3.5
Deformed bar	STD 345*2	D25	353	121.8	173.0	507	174.9	248.4	3.98
	STD 390	D22	303	117.7	169.7	387	151.0	216.7	3.04
Screw Jointed deformed bar	SD 295A	D22	Same as for Raw materials			387	114.2	170.3	3.04
	SD 345*2	D25	Same as for Raw materials			507	174.9	248.4	3.98
	SD 490	D22	Same as for Raw materials			387	189.6	239.9	3.04
		D29	Same as for Raw materials			642	314.6	398.0	5.04
	SD 585	D25	Same as for Raw materials			507	296.6	405.6	3.98
	SD 700	D19	Same as for Raw materials			286.5	196.3	236.4	2.25
Steel pipe expansion type	SS 1232*3	37T2	Same as for Raw materials			327	122.6	136.0	3.00
		37T3	Same as for Raw materials			481	180.4	200.1	4.00
	STKM12A	37T2	Same as for Raw materials			327	120.0	140.0	2.81
		37T3	Same as for Raw materials			481	180.0	210.0	3.98
	SS 400*4	36T2.0	Same as for Raw materials			337.5	120.0	140.0	2.65
	SS 490*4	36T2.2	Same as for Raw materials			357.9	180.0	190.0	2.76

*1 Effective cross-section area of the threaded portion
*2 JISM 2506-1992
*3 Swedish industrial specification
*4 Corrosion-resistant alloy-plated steel plate.

2.3.4 Steel Ribs

(1) General

Since steel ribs, which are arch-shaped reinforcements fitted against excavated tunnel surfaces, should take effect as soon as they are installed, they contribute to stabilizing the excavated tunnel face until the strength of shotcrete is fully developed. They also facilitate stabilization of the tunnel by forming a tight bond by fusing into shotcrete. The function and effect of steel ribs are summarized in Table 2 21. In designing steel rib, the shape and size of the tunnel diameter, the method of tunnel excavation, the stability of the tunnel face, the loads from the surrounding earth, and the allowance for the surface ground settlement must be considered so that details of the design such as dimensions, materials, shape, and spacing shall be determined to make sure the tunnel is stable. Since the steel ribs are usually used in combination with shotcrete and rock bolts, the estimation for supporting capability of the overall tunnel support system as well as steel ribs should be taken into account for efficient design. Since the effectiveness of the steel rib varies depending upon the rock condition, its applicability needs to be evaluated regarding several items including geological conditions as follows.

1) Hard rock and soft rocks: Steel rib can be applied to hard thru intermediate rocks with numerous joints and soil for the purpose of preventing the surrounding rocks from spalling until shotcrete and rock bolts have taken effect.

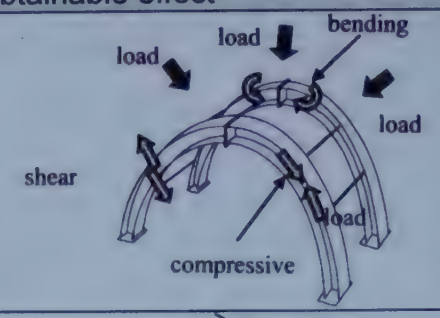
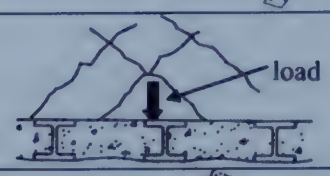
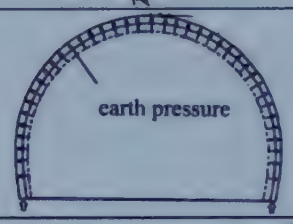
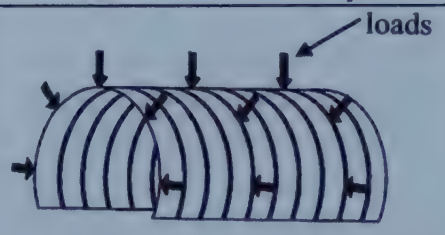
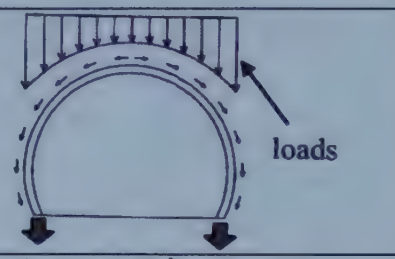
2) Soft rocks, soils, and squeezing soil: For the rocks with a low qu/P_v ratio that indicates a high possibility of failures at excavated tunnel faces, soft ground yielding plastic earth thrust, the earth that collapses quickly and squeezing rocks causing excessive earth pressures and large earth deformation, steel ribs shall be designed to improve the support strength by its integration into shotcrete which facilitates the enhancement of support of the entire rock mass by providing inner pressure along the tunnel wall, also transfer the loads acting on the tunnel support to the footing of steel rib.

The use of steel rib in combination with shotcrete and wire mesh can strengthen the stiffness and toughness of tunnel supports, which leads to safe excavation in rock masses that are likely to collapse and swelling rock masses. In addition, it enables control of tunnel deformation and surface ground settlement in the portal area with shallow overburden and the soft rocks where forming an earth arch above the tunnel is difficult by providing reaction forces along the inside of the tunnel to induce inner pressures that improve load-carrying capacity of the entire rock mass.

Forepoling with bolts or pipes with a length of 5 meters or less, or steel pipe forepiling utilizing long steel pipes may be applied in the earth where the tunnel face becomes unstable, to reinforce the earth ahead of the face in advance as shown in Figure 2-6. In the case of fore piling, since the steel rib might function as the reaction point for inserting supports in the earth ahead of the tunnel face, it is necessary to examine excessive stresses in the steel ribs according to the extent of forepiling. Since an enlargement of the section might be required according to the angle of forepiling, attention should be given to the size or shape of the steel rib.

Furthermore, buckling of steel members or failure at joints should be carefully examined when designing steel ribs.

Table 2-21: Concepts of Function and Effect of Steel ribs

Function or Effect			Description of the function and obtainable effect	
Function	I	Axial compressive resistance. Shear Resistance Bending resistance	Similar to shotcrete, steel ribs can resist external forces with compressive resistance, shear resistance, and bending resistance. Those functions are effective immediately after installation. When the shotcrete reaches an effective strength, the resistance supplied by the steel ribs unites with the shotcrete.	
Effect	1)	Support of rock mass	Steel ribs can be tightly attached to the tunnel to prevent spalling of partial rock blocks due to bending or shear resistance.	
	2)	Reinforcement of weak ground	Steel ribs can reduce the effect of discontinuities and weak layers by supporting cracks and openings or small-scale weak layers.	
	3)	Utilization of inner pressure toward the ground.	Steel ribs can provide inner pressures along the tunnel wall in the radial direction for weak ground that cannot be expected to form a ground arch. This allows a confined stress state that increases load-carrying capacity of the entire ground.	
	4)	Reinforcement of shotcrete	Steel ribs fuse with the shotcrete to improve the stiffness and toughness of shotcrete with low elastic modulus and strength. In addition, after the appearance of sufficient strength in the shotcrete steel, steel rib can be integrated into the shotcrete and applied tightly on the tunnel wall to form an arch shell structure and then stabilize the tunnel and the surroundings.	
	5)	Transfer of loads to the ground (footing)	Steel ribs can transfer the loads from the surroundings to the foundation through the wing ribs.	
	6)	Support for forepoling	Steel ribs can function as the reaction point for the fore poling supporting the ground ahead of the tunnel face and restrain the failure or loosening of the surrounding ground.	

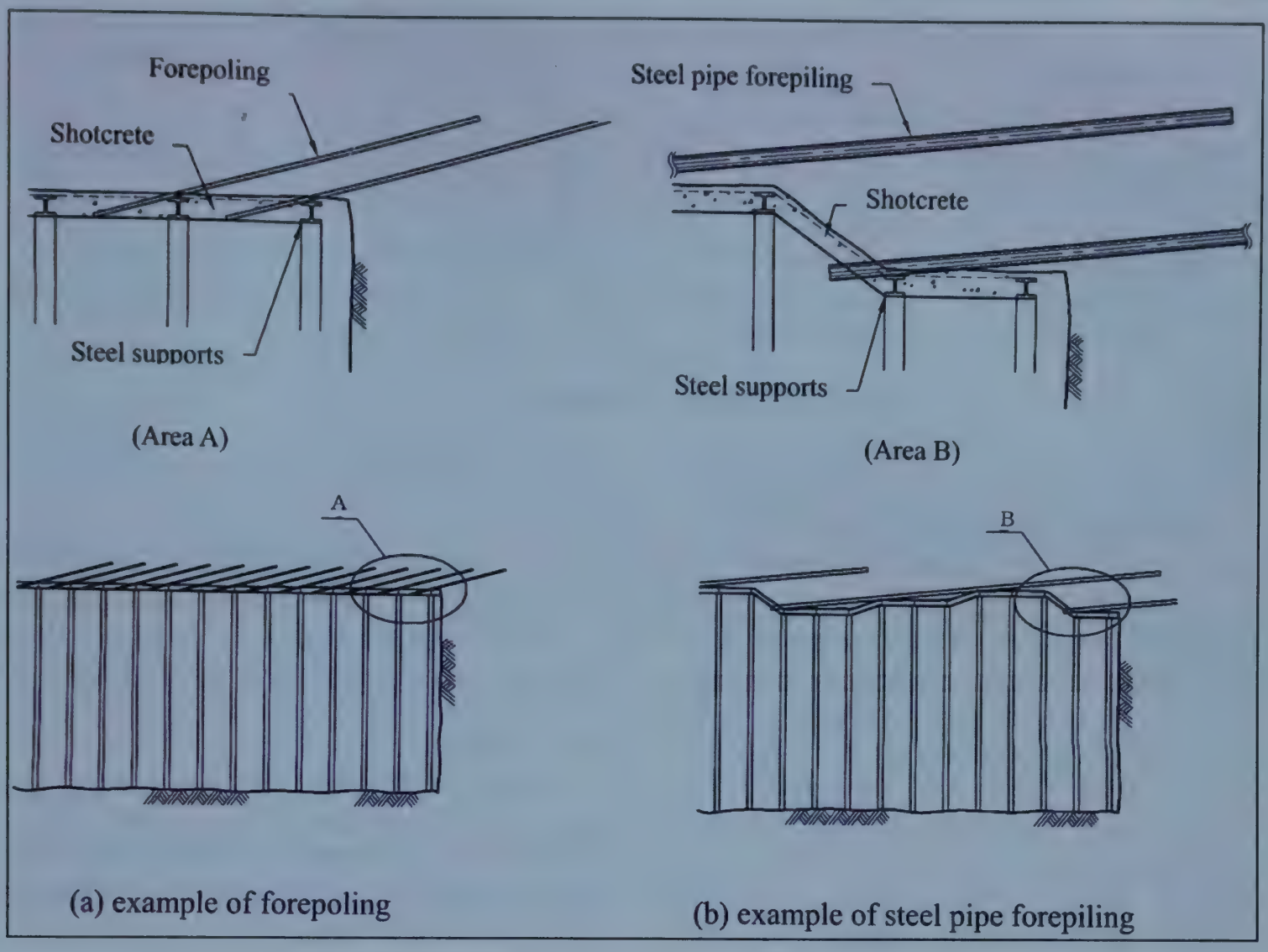


Figure 2-6:Utilization of Steel ribs as Reaction Supports of Pre support

(2) The shape of Steel Ribs

Steel ribs should be designed to bear the loads properly transferred through the shotcrete with yield bending moment as small as possible in a shape similar to the tunnel shape. The shape of the steel rib includes the top heading, upper and lower halves, and all surroundings as shown in Figure 2-7, and should be determined based on the nature of the surrounding rocks, intensity and direction of applied load, and installation method

teel supports are generally a circular arc shape, and any shape which may cause excessive stress in the earth or tunnel support must be avoided as much as possible. However, the footing area of steel ribs may be expanded by either enlargement of the base plates or employment of the wing ribs so that its bearing capacity can be improved to control the settlement of tunnel support.

Moreover, regarding the use of wings ribs, particular attention should be paid to the design seeing that the wing rib method is supposed to complicate and enlarge the shape of the excavated section, which may cause disturbance of the surrounding rocks resulting in lowering of stability, as well as the increase of the total weight of steel ribs which may reduce working efficiency.

Moreover, since the steel ribs are integrated into shotcrete and attached tightly to the tunnel wall, they can form an arch shell structure and stabilize the tunnel and surrounding rocks. If the condition of the rocks is good and the tunnel can be stabilized without having to be concerned about controlling settlement with a steel rib footing, the lower half of the steel rib can be omitted.

Sufficient margin shall be allowed in designed manufacturing dimensions, given tunnel deformation and errors during manufacturing and assembly.

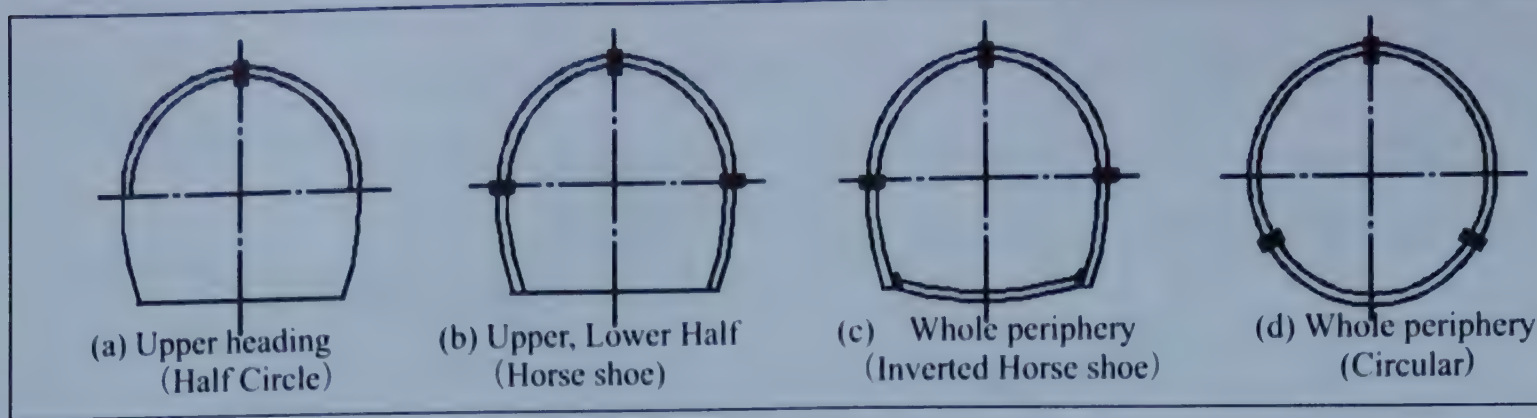


Figure 2-7:Various Shapes of Steel ribs

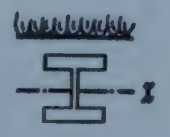
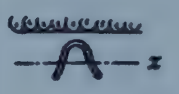
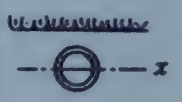
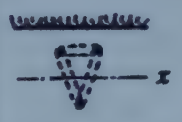
(3) Shape and Quality of Steel Ribs

1) The steel used in steel ribs should be selected to have proper strength and stiffness with reference to the active load, thickness, and work method of shotcrete, tunnel dimensions, and excavation method. It is highly recommended that the shape of steel ribs be determined so that they are easily integrated into the shotcrete and applied tightly to the tunnel wall. Moreover, it is desirable, under large loads, for the steel rib to be designed strong enough to resist buckling and torsion.

The steel used for steel ribs usually includes H-beams, U-beams, circular pipes, and lattice girders. Currently, H-beams are most frequently used. The lattice girder, which has a truss shape consisting of three deformed reinforcing bars may combine well with shotcrete. However, it produces a somewhat large deflection due to its low stiffness compared to other steel products.

2) The material for steel ribs does not suffer brittle failure even when large strains are expected under large loads. The material should be well applicable to the precise processing of bending and welding of joints. Usually, SS400 is used effectively. For a large diameter of highway tunnel possessing three lanes, the high standard (HT590/SS540) is employed to decrease the weight. Although cold processing is recommendable because of its excellent quality control, particular attention is required to sectional and torsional deformation during processing. Table 2 22 lists the parameters of steel products used in steel ribs.

Table 2-22: Parameters of Steel Products for Steel ribs

Type	Material	Dimension (mm)	Sectional Area A (cm^2)	Density W (kg/m)	Moment of Inertia Ix (cm^4)	Section Modulus Zx (cm^3)	Minimum Radius of Curvature R (cm)	Illustration of Use
H-beam	SS400	H-100×100×6×8	21.59	16.9	378	75.6	120	
		H-125×125×6.5×9	30.00	23.6	839	134	150	
		H-150×150×7×10	39.65	31.1	1 620	216	200	
		H-175×175×7.5×11	51.42	40.4	2 900	331	340	
		H-200×200×8×12	63.53	49.9	4 720	472	420	
		H-250×250×9×14	91.43	71.8	10 700	860	550	
	HT590/ SS540	H-154×151×8×12	47.19	37.0	2 000	260	400	
		H-200×201×9×12	65.53	51.4	4 782	478	450	
U-beam	SM490	MU-21	26.80	21.0	296	56.6	135	
		MU-29	37.00	29.0	581	97.4	150	
Steel Pipe	STK400	φ76.3×3.2	7.35	5.8	49.2	12.9	65	
		φ89.1×3.2	8.64	6.8	79.8	17.9	80	
		φ101.6×5.0	15.17	11.9	117	34.9	100	
		φ114.3×4.5	15.52	12.2	234	41.0	120	
		φ139.8×6.0	25.22	19.8	566	80.2	140	
		φ216.3×8.2	53.61	42.1	2 190	269	250	
		φ267.4×9.3	75.41	59.2	6 290	470	450	
Re-bars	SD295	D16×3+D10	6.00	7.4	94.4	26.2	-	

* Values of minimum radius of curvature R are by means of the cold processes

(4) Spacing of Steel Ribs

The spacing of the steel ribs shall be determined considering face stability, the intensity of earth pressure, sectional dimensions, intended purpose, excavation method, tunnel driving method, and other factors. Spacing of 150cm or less is assumed

(5) Joints and Base Plates for Steel Ribs

Since the joints of the steel ribs may become a structural weak point of the support members, it is necessary to design joint positions and connected mechanisms considering the excavation shape, as well as the size and distribution of stress. Beams should be strongly connected because buckling and destruction due to bending may occur at the joints. It is necessary to acquire sufficient load capacity for the steel ribs by not only enlarging the joint plates and base plates but also by integrating them securely into the shotcrete to keep the settlement as small as possible.

Figure 2-8 illustrates an example of joints and base plates for steel ribs.

If the joint plate protrudes toward the lining concrete, the thickness of the lining is inadequate which weakens it or damages the waterproof sheet on the lining. It is, thus necessary to keep the joint plate as far as possible inside the shotcrete. Likewise, the base plate of the steel rib should be kept inside the shotcrete only for the upper heading, to avoid the problems mentioned above.

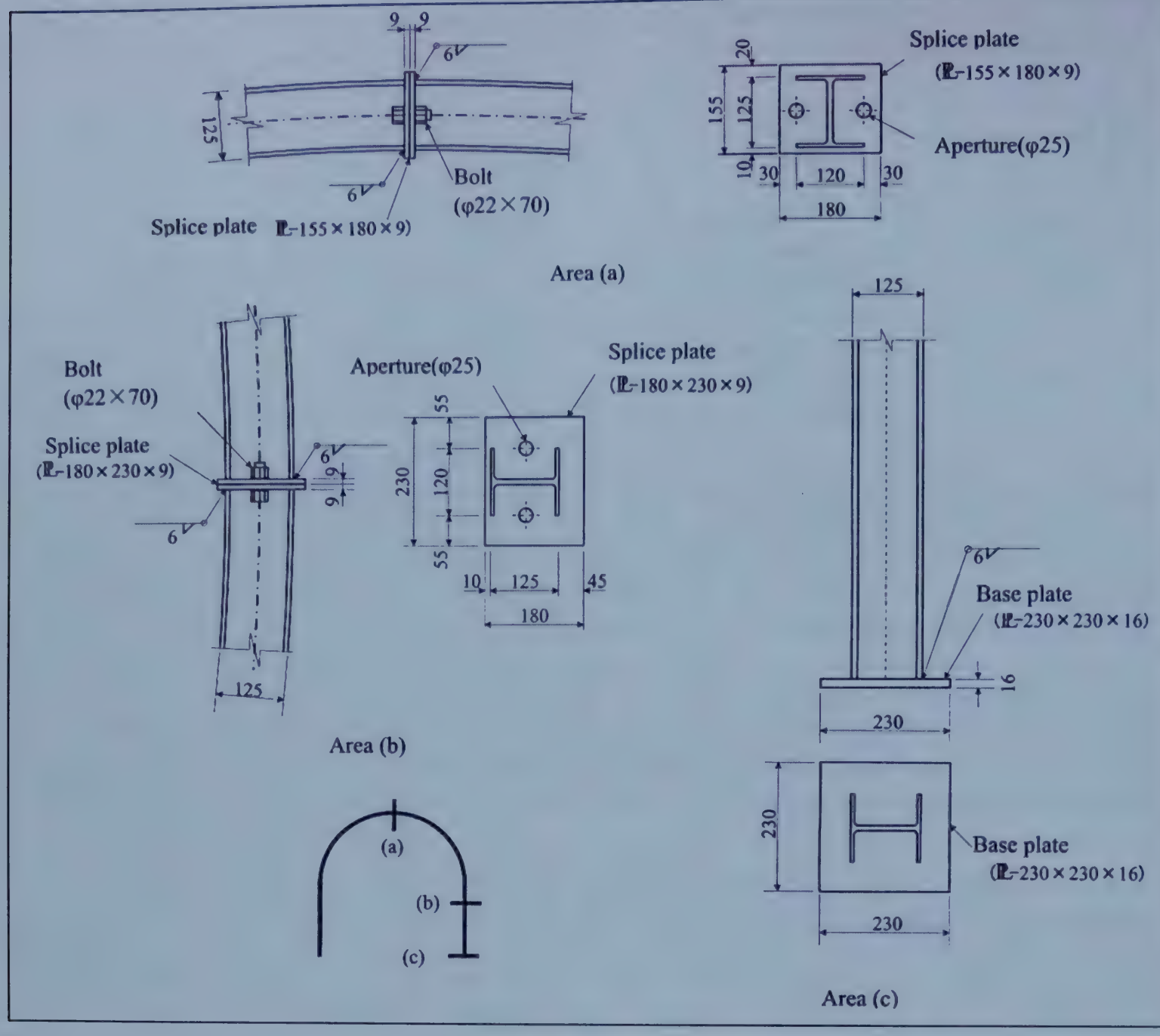


Figure 2-8:Examples of Joint and Base Plate

(6) Collar Braces for Steel Ribs

Collar braces shall be properly installed on the newly erected steel ribs to prevent collapse until the support is fixed in place by shotcrete. Moreover, the strength of steel ribs shall be examined in cases where an external force is anticipated in the direction of the tunnel. In addition, a structure to prevent voids from forming behind the joints while shotcrete is required.

There are two types of collar braces as shown in Figure 2-9, type-(a) sheath tube method has been generally utilized, whereas type-(b) tie rod method is mostly used at the portal area and the sections without shotcrete.

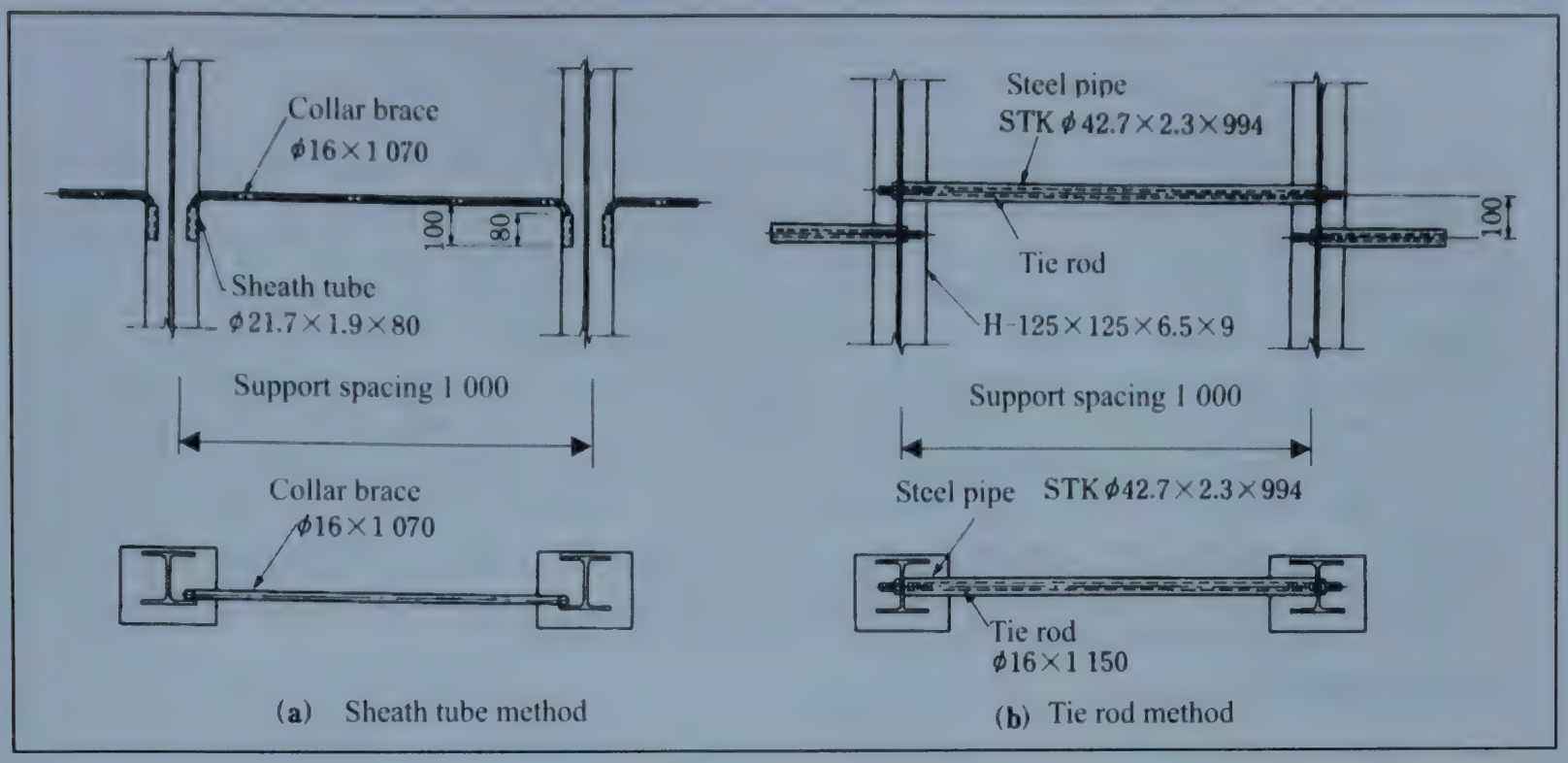


Figure 2-9: Examples of Collar Brace

2.3.5 Lining

(1) General

Considering that a tunnel lining is very difficult to repair while in service, it should be designed with sufficient consideration for long-term durability to make tunnel lining free from repair as long as possible. Since the tunnel lining is a thin arch-shaped concrete structure that is installed on the narrow space between lining forms and the inner surface of the tunnel, which means that the working conditions are too poor to place and compact the concrete adequately, the concrete lining is apt to suffer cold joints, cracks, separation, cavities including openings behind the lining wall, damage to the waterproof sheet, lack of thickness, rough surfaces, and so forth. Therefore, the lining should be designed to satisfy the requirements identified based on such possible problems. Moreover, the design of the tunnel lining must sufficiently account for rock conditions, load conditions, facility importance, and other conditions. It should be kept in mind that the tunnel lining serves the following purposes.

For serviceability,

1) To decrease water leaks (e.g. groundwater) and improve water tightness.
2) To improve the efficiency of inspection and repair for the tunnel in service.
3) To efficiently use water tunnels by better control of coefficient of roughness for water tunnels.
4) To provide essential facilities such as overhead wiring, lightning, and ventilation.

For strength characteristics,

1) To improve safety for the entire structure for uncertain factors such as geological heterogeneity, and non-uniform quality of tunnel supports. Since no load is applied to the tunnel lining which is the same as rocks, tunnel supports help settle displacement.
2) To provide the restraining force required to stabilize tunnels where the tunnel lining is the same as rocks, and displacement has not yet settled.
3) To support external force after completion of the tunnel lining that could be induced by water pressure or extra loads.
4) To improve structural durability against variations in external loads, deterioration of tunnel supports or surrounding rocks after the tunnel is put in service.

Tunnel inverts are merged into tunnel supports and form a ring structure, which facilitates the stabilization of the surrounding rocks. The inverts are effectively merged into tunnel supports and lining for the following conditions;

1) For tunnels in urban areas, tunnels in weak rocks, tunnels in soil which is likely to significantly lose their load-carrying capacity over time due to deterioration or weathering of the rocks.
2) For tunnels with load conditions likely to change in future
3) For tunnels in fault fractured zones necessitating earthquake resistance
4) For tunnels in portal areas that are subject to unsymmetrical loads and earthquakes
5) For railways tunnels with the high possibility of mud pumping

(2) Principle of Lining Design

There are two different philosophies for designing tunnel lining, to impose mechanical function on the lining or not.

For most mountain tunnels, the lining is installed after the rock displacement has completely settled with the help of tunnel supports. For that reason, the lining is usually intended to serve without mechanical function on the assumption that no external load acts on it. In this case, the lining intended mainly for serviceability shall be designed and built on the basis of un-reinforced concrete. Since the serviceability is generally impossible to quantitatively estimate as a design item, the thickness of the lining prescribed in the Design Standard is applied. If the lining is placed based on the prescribed design thickness under adequate work control, it can be relied on to achieve the general function as a tunnel lining including serviceability and the desired qualities including long-term durability.

Regarding mountain tunnels, the following cases involve mechanical function in the design of lining.

1) For changes in tunnel conditions such as embankment or cutting, or additional loads acting on the lining due to parallel tunnels or structures in the vicinity of the tunnel.
2) For lining subjected to continuing displacement in poor rocks such as expansive rock.
3) For lining subjected to external hydraulic pressure

Where the lining is designed to perform mechanical functions, it is necessary to enhance the load-carrying capacity of the lining through the utilization of reinforced concrete or steel fibre reinforced concrete, and installation of invert to encase the lining enclosed against external loads. Some analysis methods make it possible to determine the thickness, strength, and amount of steel reinforcement. Even fibre reinforcement is included against separation and stripping rather than enhancement of load-carrying capacity.

Although the design of the lining is mostly established following the allowable stress design method, the limit state design method has been also introduced for the lining design.

Even though precise estimation of the load acting on the lining can provide the adequate design load for analysis of the lining serving as a mechanical member in conjunction with safety corresponding to the purpose of tunnel and surrounding conditions, no trustworthy monitored data for the load (earth pressure) acting on the lining in mountain tunnels have been accumulated so far, which indicates extremely low reliability of the current data relating to design load for the lining.

(3) Shape of Lining

Generally, the arch-type of lining shall be used, applied to the desired shape of the excavation site, and designed with a loading capacity to accommodate such active loads as earth pressure.

Where the lining is designed as an arch in combination with simple curves, curves with multiple curvatures such as curves of triple-curvature, curves with five curvatures, and straight lines, it is desirable to locate such circles, arcs, and lines to share common tangential points at their connections for the formation of smooth arches. Because lining with sharp curves or corners increases the bending moment due to the eccentricity of axial load, as does the lining with a long straight line due to the long span prone to excessive deflection, these shapes should be avoided.

Figure 2-10 shows the variations of lining shapes. The one on the far right is most suitable for poor rocks. For good rocks, combinations of arches and straight lines, and slightly curved lines shall be enough to form a lining shape, whereas, for poor rock, an invert is included in the sectional shape.

If needed the tunnel connected to shafts or the cross passages should be adequately reinforced where the shape differs from the typical shape. The thickness of the lining should be kept thick enough to avoid loss of stiffness particularly for sections that change shape abruptly such as concave areas for storage of electric equipment.

Many large-diameter tunnels such as road tunnels with three lanes, enlarged or diverging sections may employ flat shapes rather than the shape with the best structural performance. Given that the flat-lining may produce some excessive bending moments depending on rock condition, the thickness, strength, and amount of reinforcement of lining should be properly estimated.

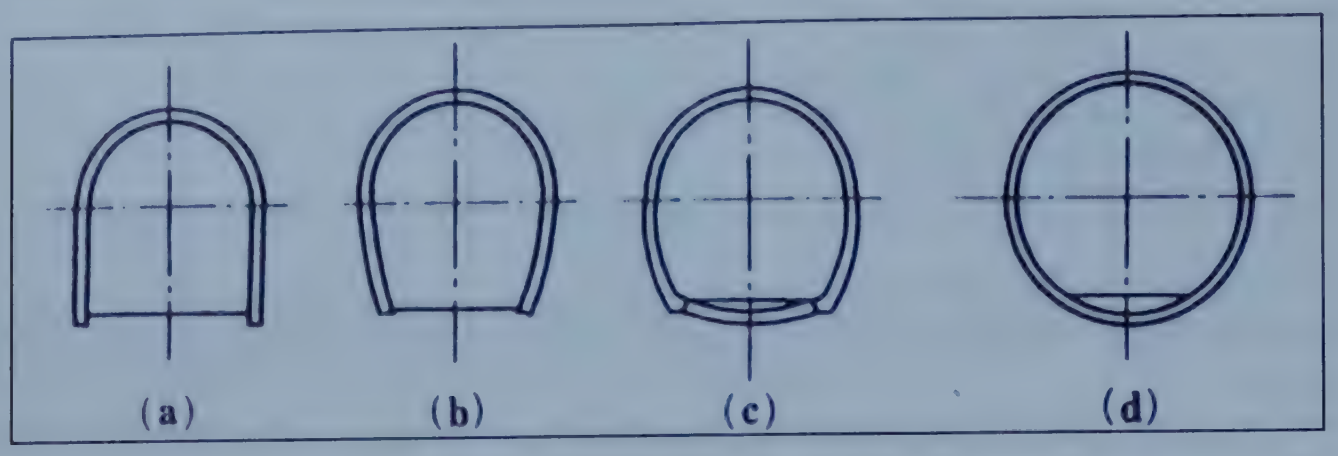


Figure 2-10: Shape of Lining depending on Rock properties

(4) Design Thickness of Lining

The standard design thickness of the lining is readily determined based on the lining function, installation method, and other factors. The standard design thickness enables the lining to achieve the required functions under adequate work control unless the tunnel passes through extremely weak rocks or shallow overburden. The following things are employed to improve the lining;-

- to make lining a mechanically advantageous shape
- to apply reinforced concrete to improve the bending strength of concrete
- to apply high strength concrete to improve the bending strength of concrete
- to apply fibre reinforced concrete to improve the bending toughness of concrete

The lining shall be designed with sufficient consideration for the effects of large and flat tunnels, urban tunnels, tunnels close to the existing tunnel.

(5) Mix and Strength of Lining Concrete

The desired strength of lining concrete varies depending upon such conditions as the characteristics of the rocks, shape of the lining, type of support, and load applied to the lining. The specified concrete strength is often adjusted to about 18 to 24 N/mm2 except in special cases. The design engineer shall design the unit cement content, water content ratio, slump, and other factors considering the materials and work conditions, in addition to the desired strength mentioned above.

Moreover, given that the lining concrete is placed using the concrete pump, the mixing of concrete with fixed pumpability is needed for excellent working efficiency. Since pumpability is dictated by the workability and consistency of concrete, the mix of lining concrete should be designed by considering the discharge determined form these factors, the water-cement ratio determined from the strength, and by minimizing the heat generated by hydration. In addition, to these conditions, work conditions such as stripping time and conveying time of concrete are also necessary to be included in the mix design for lining concrete.

Special measures should be taken where such harmful elements as salinity and organisms are present in spring water and the rocks, which adversely affect the concrete mix.

(6) Prevention Measures against Cracks in Lining Concrete

Although a concrete lining is apt to undergo cracks caused by materials, circumstances, and construction, the prevention measures to eliminate the causes of cracks and the remedies for materials and construction could reduce the cracks. Moreover, not all

cracks are detrimental to lining concrete. Some cracks do not affect the function of lining because of their width and shape.
Cracks in lining concrete significantly reduce the strength characteristics and water tightness of the lining, and adversely affect the durability, safety, and the original functions of lining due to water leakage, the formation of icicles, freezing, and thawing. In addition, some concrete surrounded by multiple cracks intersecting together in a three-dimensional space might separate and spall. Therefore, it is necessary to examine carefully which cracks have a harmful effect on the lining in conjunction with consideration to the function of the lining.
The length of the lining is much greater than the thickness with respect to both the direction of the tunnel axis and circumference and the outer surface of the lining bordering the tunnel is restrained by the previous shotcrete. Apart from the cracks in lining concrete caused by deformation due to excessive loads, it can be said from the geometry and boundary condition of the lining that the cracks are easily formed by the tensile stresses due to the shrinkage stress controlled by the lining concrete.
The main factors causing the stress from shrinkage in lining concrete are as follows;

1) Shrinkage due to a drop in concrete hardening temperature
2) Shrinkage due to a drop in tunnel temperature
3) Drying shrinkage due to low humidity in tunnel
4) Shrinkage during hardening

However, not all these types of shrinkage stress necessarily become harmful.

Prevention measures against cracks are as follows;

1) Isolating with shotcrete (reduction of external restraint)
2) Improvement of concrete quality and materials (decrease in shrinkage stress or increase in tensile strength)
3) Placement of crack inducing joints (control of crack formation)
4) Improvement of concrete placing environment

The isolation of the shotcrete and lining concrete is considered the most effective method for reducing the stress between shotcrete and the outer surface of the concrete lining. The method of placing isolation sheets or waterproof sheets is the most commonly used method.
Where there is extensive cracking, the quality of the concrete and the effectiveness of materials need to be examined. The use of adequate expansive admixtures of shrinkage reducing agents is highly recommended for controlling cracks induced by shrinkage of concrete.
Crack-inducing joints provide a means of controlling the location and direction of newly generated cracks and reducing the formation of cracks by placing a joint in a specific location beforehand. These joints also facilitate treatment after cracking.
Since cracks can be reduced by the restraining effect of an invert, aligning the joints in the lining to correspond with joints in the invert is recommended.

2.3.6 Invert

(1) General

Considering that inverts are extremely difficult to repair after the tunnel is put in service, it is necessary to design the invert to stabilize the surrounding rocks by merging into the adjacent tunnel supports, and then achieve a satisfactory role as a permanent structure carefully considering rock conditions, site conditions, the purpose of use and necessary functions and quality.

1) Invert functions
   Inverts are recommended to provide the following functions;
   For serviceability,

   (i) Function to hold the necessary shape together with the lining

   (ii) Function to provide smooth flow path together with the lining

   For mechanical characteristics

   (i) Function to prevent settlement due to lack of bearing capacity in case of poor rocks or displacement of the side wall of tunnel induced by plastic earth pressure.

   (ii) Function to enhance structural stability by forming a ring-shaped structure in combination with lining to achieve sufficient load-carrying capacity against earth and water pressures that are exerted on tunnels in urban areas for a long period.

   (iii) Function to improve the durability of tunnels against deformation due to heaving in the rocks that are likely to undergo weathering or swelling, or deterioration due to application of repeated loads before and after construction.

   (iv) Function to improve stability against structural deformation of the tunnel by early formation of a ring-like structure which combines with tunnel supports to control convergence of the tunnel during construction similar to temporary invert placed by shotcrete

2) Invert placement
   Inverts are placed at portal areas, poor rocks including soft soil, and areas liable to loosening for a long period due to weathering right after excavation.
   Basically road tunnels employ inverts for portal areas, every rock mass corresponding to D in rock mass classification, and mudstone or tertiary period, tuff, serpentinite, weathered crystal schist corresponding to C in rock mass classification, soils susceptible to slurry-like such as solfataric soil clayey soil.
   Incidentally, railway tunnels built by any means employ inverts for rocks corresponding to I_N of less, mudstone corresponding to II_N, hard rock likely to spall (Type B).
   Moreover, inverts should be employed in tunnels in urban areas, which are close to other tunnels, likely to suffer damage due to seismic loads in an account of the existence of a fault fractured zone, subject to change of loading condition due to shallow overburden and earth projects above them. Where inundation of the bed in a tunnel is anticipated on account of the use of heavy construction equipment for excavation and mucking, installation of an invert at an early stage is recommended.
   For poor rocks, the invert should be placed as soon as possible after excavation to enclose the tunnel completely, which leads to stabilization of the tunnel.

(2) Shape and Thickness of Inverts

Inverts resist external forces with the arch and sidewall and stabilize the entire tunnel lining. The shape and the thickness of the invert are usually designed based on past tunnelling experience. However, it is preferable to determine each factor separately in

the case of a large tunnel, strong earth pressure acting in the tunnel, or soil with shallow overburden such as urban tunnels.
For road tunnels, the thickness of inverts is prescribed as part of the standard tunnel support system according to the gconstrurade of rocks. It is determined for railway tunnels and water tunnels according to the grade of rocks in terms of standard design thickness as shown in Table 2-23 and Table2-24.
An example of invert structures is displayed in Figure 2-11 and Figure 2-12.

Table 2-23: Standard Thickness of Invert for Railway Tunnel

(For conventional lines of JR and Shinkansen – bullet train)

Grade of ground	II_N	I_N	I_S	I_L
Thickness (cm)	Properly	45	Shotcrete 15 30	45

Table 2-24: Standard Thickness of Invert for Water Tunnel

Tunnel Diameter	Design Thickness (cm)			
	Crown Arch / Side Wall		Invert	
	Type A & B	Type C, D	Type A & B	Type C & D
< 3.0 m	15	15	15	20 (15)
≥ 3.0 m	20	20	20	25 (20)

1) The thickness of invert for Type C & D can be adjusted to ( ) if invert shotcrete is placed or no evidence of swelling is found.
2) The values in this table are thickness from the inner surface of the lining

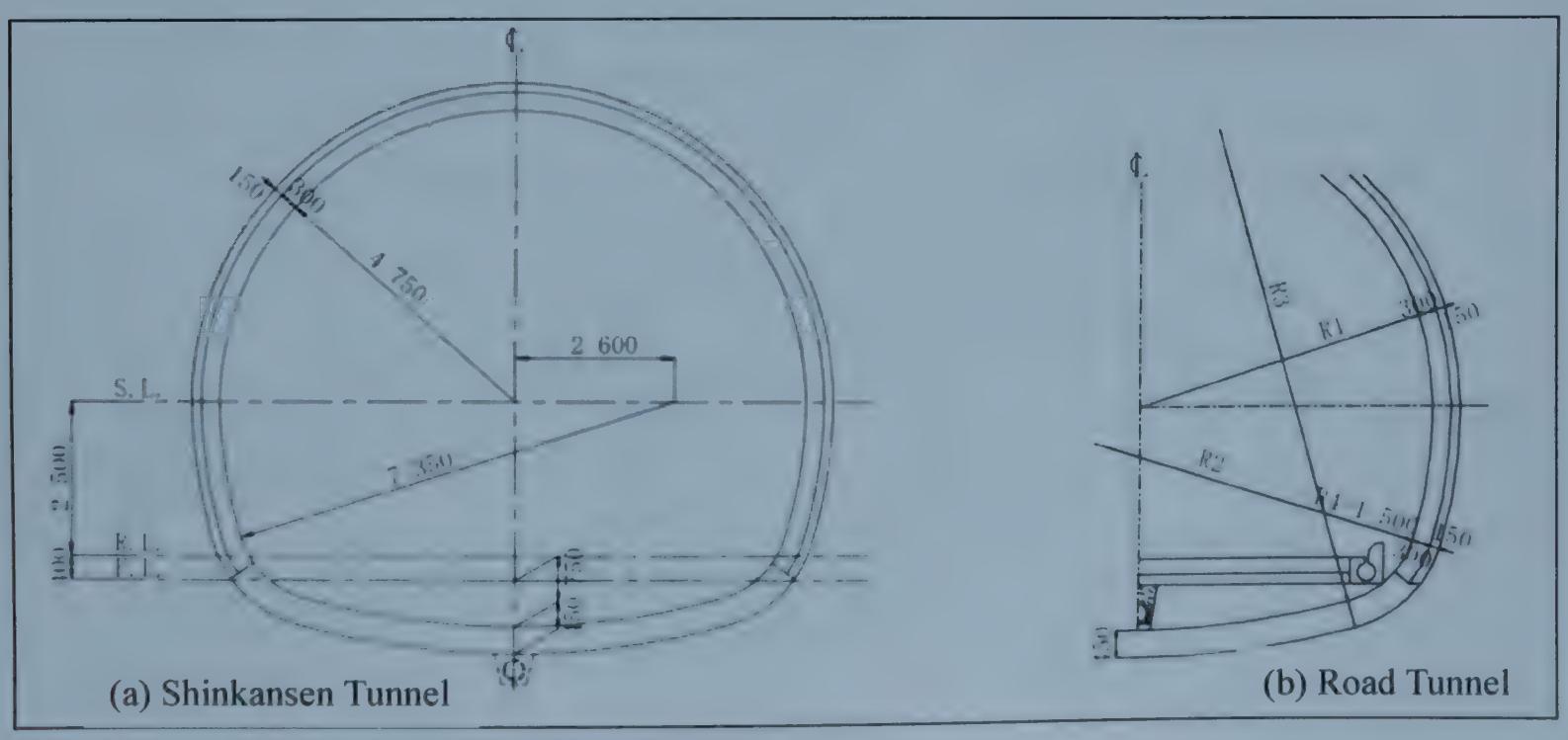
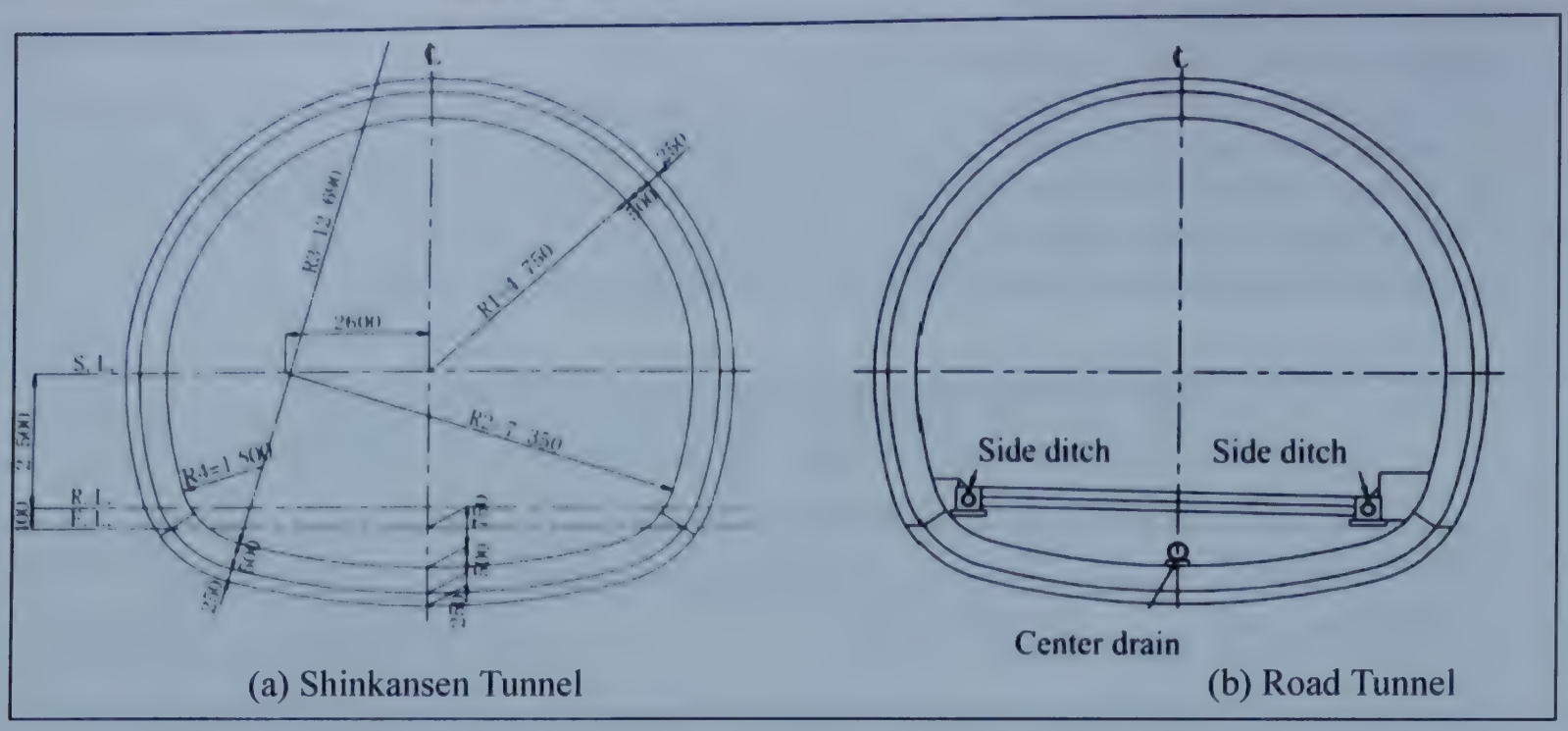


Figure 2-11: Example of Standard Invert



(a) Shinkansen Tunnel (b) Road Tunnel

Figure 2-12: Example of Invert placed in a Mountain Tunnel in an Urban Area

2.3.7 Waterproofing and Drainage Systems

(1) General

Water leakage in tunnels not only degrades the functions of linings and in-tunnel facilities and reduces their durability but also has an adverse effect on travel on the road in the tunnel due to the road surface freezing and the freezing of water in winter. Appropriate waterproofing and drainage systems therefore should be designed.

During the excavation of mountain tunnels, groundwater surrounding the tunnels is generally drained without being allowed to remain behind the lining, to control extremely high groundwater pressure or prevent water leakage through the lining. When tunnelling is in the soil in urban areas, however, plans should be made to design and construct so-called watertight tunnels that cause groundwater to condense after tunnelling. Waterproofing and drainage systems should be designed separately for ordinary and watertight tunnels.

As shown in Figure 2-13, in many instances the position of drain pipe with invert is inside the tunnel (above the invert) for a highway tunnel and outside the tunnel (below the invert) for a railway tunnel.

Generally, control measures for inflow water include i) waterproofing by waterproof membranes after taking appropriate measures behind the lining to prevent water leakage inside the lining, ii) drainage system such as back surface drainage materials and side drains to carry away water flowing into the tunnel and prevent standing water or iii) control of water leakage through the construction joints by cutting off or conveying the water despite the application of waterproofing. The drainage methods are shown in Figure 2-13.

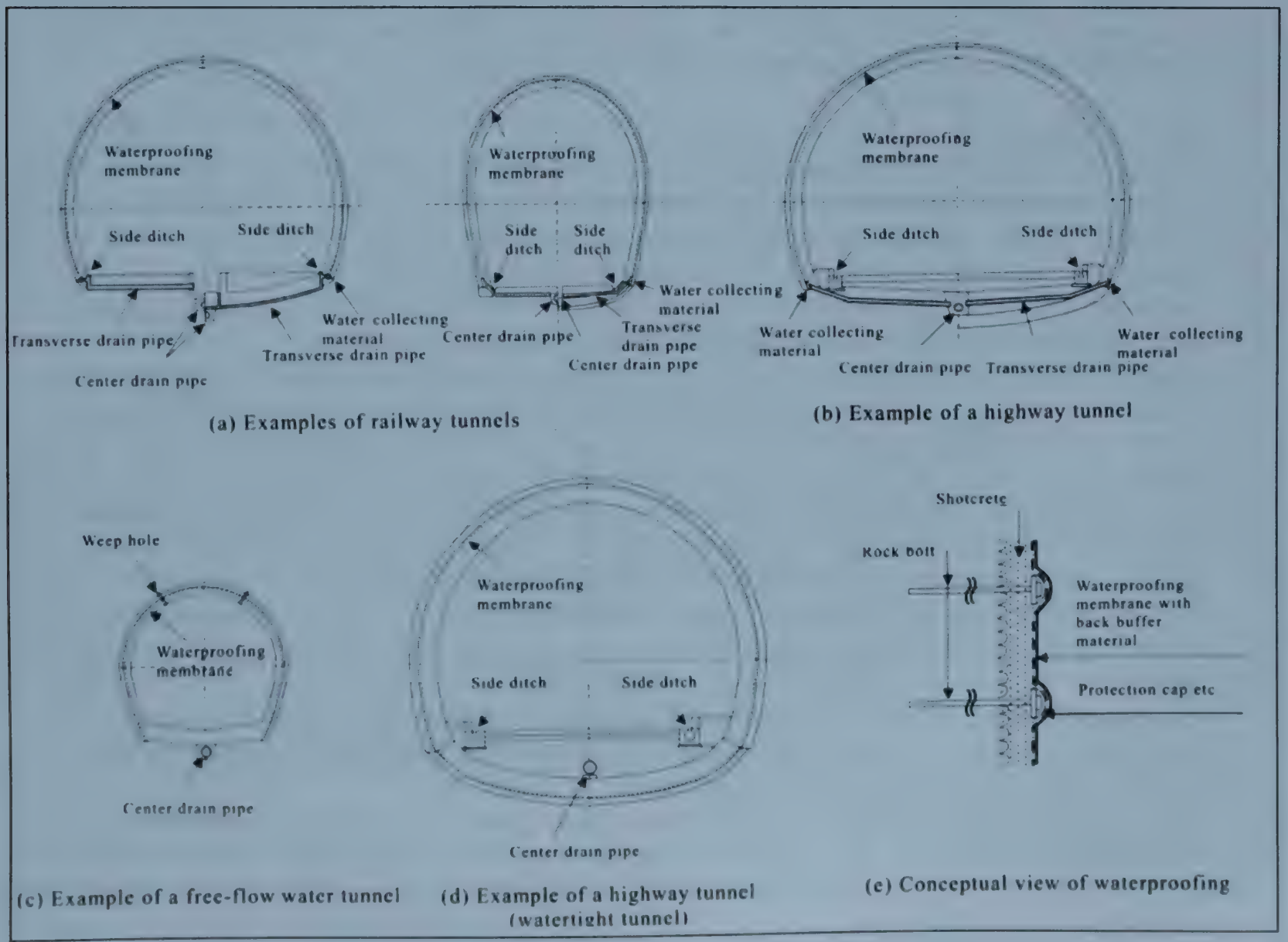


Figure 2-13: Example of Waterproofing and Drainage

(2) Waterproofing

An appropriate waterproofing system should be designed based on the rock condition and the use of the tunnel because water leakage in the tunnel makes travel on the roadway difficult due to road surface freezing and the freezing of the water in winter. In many cases, leakage also deteriorates the durability of the lining and has an adverse effect on the facilities in the tunnel. The applicable area of the waterproofing system should be determined carefully because even an area with no apparent water inflow suffers leakage after the completion of the lining because of changes in groundwater condition.

Three typical methods of waterproofing in mountain tunnels are the application of waterproofing membranes, spraying waterproofing, and coating with a waterproofing film. At present, waterproofing membranes are used more frequently than any other method because they can form a highly reliable waterproof layer of uniform quality. Spraying and coating can be carried out easily and fit the surface to be waterproofed satisfactorily. However, by using these two methods it is difficult to produce a waterproof layer of consistent thickness.

Materials for waterproofing systems should have sufficient expansibility and strength to resist mechanical impact during the installation of waterproofing systems, placement of lining concrete, fresh concrete pressure, and water pressure. They should also be of structure and quality to provide durability against the components of concrete and water inflow. Especially in watertight tunnels, high water pressure may act on the waterproofing system, and therefore a thorough study needs to be made of the type, material, specifications, and the applicable area of the waterproofing system. Because the formation of a continuous waterproof layer is required, joints of the waterproofing membranes in particular also need to have the designated strength and water tightness. Waterproofing materials should be very safe against fire, and superior from the standpoint of ease of installation and economic efficiency.

(3) Drainage

During the design of drainage systems, the positions of installation, structural form, section, and other aspects need to be determined considering the use, location, amount of water inflow, longitudinal and transverse gradient of the tunnel, the need of an invert, track structure and other factors. As a rule, drainage systems should have the same gradient as the roadbed, and if possible should have a structure capable of capturing and draining water inflow that gathers under the roadbed.

Water flowing into the tunnel that collects behind the lining tends to run at the lowest points in the tunnel section. When the position of the drainage system is inappropriate, water can be retained near the formation level. This can cause deterioration of the roadbed as a result of increased water pressure in cracks, mud pumping, or other action resulting from repeated loads accompanying the operations of rolling stock, freight cars, and other vehicles. Because the lowest points in a tunnel section generally are located at the centre, in many cases a drain or other measure is installed at the centre of the tunnel section. For this drainage, water-collecting material such as filter mats or mesh tubes are placed at the bottom of the sidewall between the shotcrete and the concrete lining, or a perforated pipe is laid in the earth. Transverse drain pipes, transverse drainage ditches, or other measures are installed at appropriate intervals, depending on the water inflow, to convey the water to the centre drain pipe, centre drain ditch, or side ditches. This drainage method is shown in Figure 2-14.

For road tunnels, the design engineer also must plan an appropriate gradient to drain water used for washing, fire-fighting water, and tunnel control water to the centre drain pipe or centre drain pipe and side ditches after use, so the water does not collect and stagnate.

High-density polyethylene pipes and hard vinyl chloride pipes are used as materials for centre drain pipes and transverse drain pipes. For backfill material when perforated

pipes are used, backfill material with an appropriate grain size must be selected to prevent blockage of the drain pipe in the future.
For railway tunnels, in some cases checkboxes are installed at appropriate intervals for purposes such as inspecting and cleaning the drain pipes after use (refer to Figure 2-15).
If effective drainage is emphasized in sections where an invert is required, the installation of a drainpipe on the bottom surface of the invert is advantageous because it enables drainage at the lowest point.
A drain pipe often is installed on the upper surface of the invert in the following cases.

1) When there is an invert on the upper flow side or in the middle of the tunnel length, and considerable excavation is required because the section without an invert and the drain pipe installation height differs, and the benefit described above is thus offset.
2) Where the installation of a drainpipe under the surface of the invert can disturb swelling rocks
3) Where fine particles flow into the drainpipe with the water inflow in soil, which is likely to create voids under the surface of the invert and cause the ground surface to subside as a result.

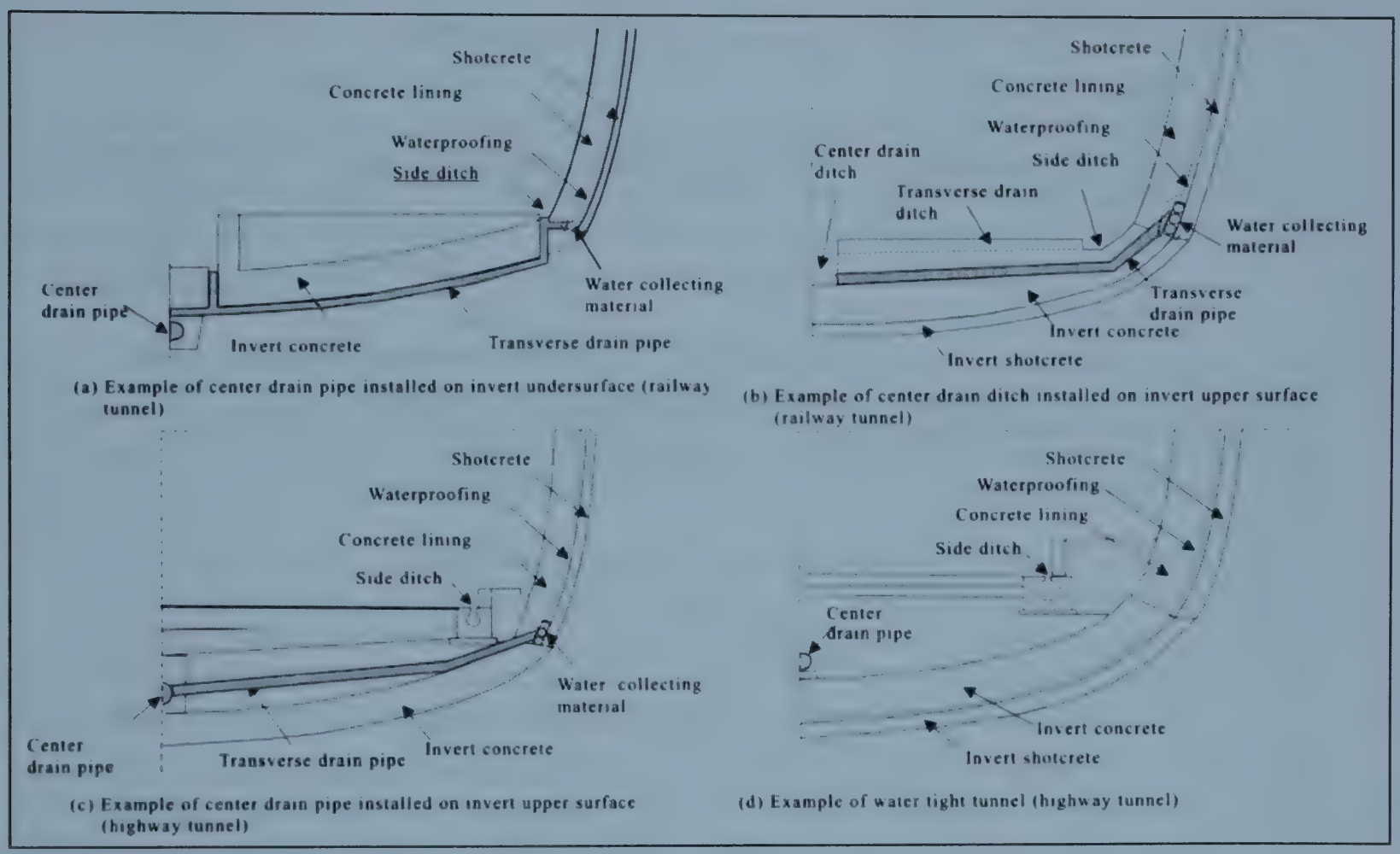


Figure 2-14: Example of Drainage Systems

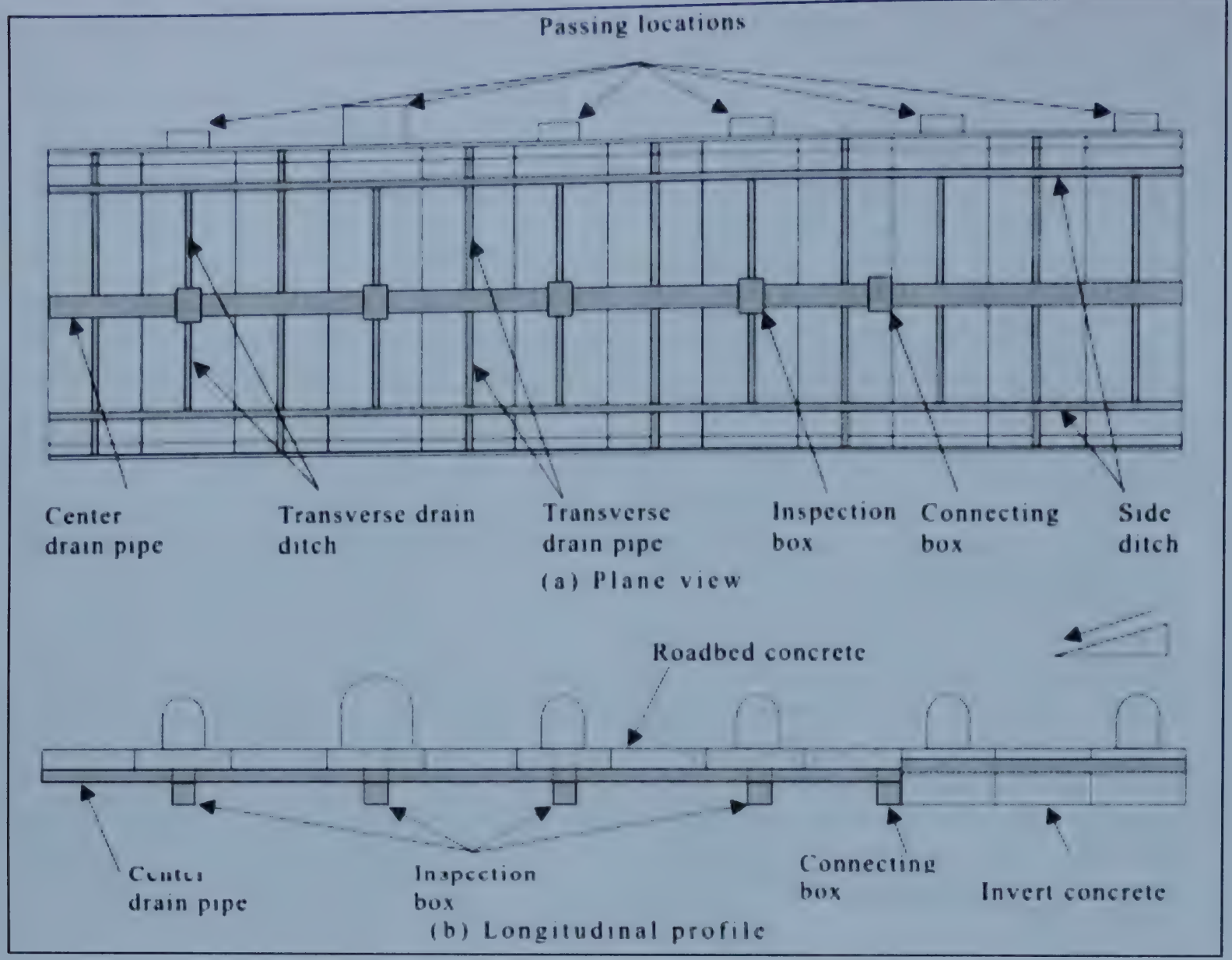


Figure 2-15: Example of Inspection Box Installation

CHAPTER 3 REINFORCEMENT WORKS

3.1 General of Reinforcement Works

Reinforcement works are a construction method of secondary or special nature adopted to ensure face stability and tunnel safety and to preserve the environment in cases where either conventional tunnel support systems or divisions of heading section do not provide effective solutions or where they are not advantageous.
Practical methods that can be used with ordinary tunnelling machines and materials, which will not have a significant impact upon construction cycles, include face stabilizing techniques such as forepiling, long forepiling, face shotcrete, face bolts, foot reinforcing bolts, temporary invert, and techniques for controlling groundwater such as weep holes. In contrast, methods that are not very practical for use with ordinary tunnelling machines and materials include measures to avoid adverse impact on structures in the vicinity such as grouting, shielding wall, and measures against surface settlement such as long forepiling, pipe roofs, and groundwater measures such as drain boring, well points, deep wells and weep holes (Table 3-1, page 3-3).

3.1.1 Role of Reinforcement Works

Since reinforcement works are closely related to tunnel design and construction methods, it is necessary to change the excavation method and tunnel support system appropriately so that they will be suited for the selected reinforcement works and their objective and type.
The major objectives of reinforcement works are ensuring the safety of tunnelling (face stabilization and measures against water inflow) and preservation of the environment (groundwater measures, prevention measures against surface settlement, and measures to prevent adverse impact on structures in the vicinity). The selection of reinforcement works should be based on a comprehensive study to meet these objectives.

1) Reinforcement works to be used during tunnelling should be selected, considering their effects, cost efficiency, and work period, correctly evaluating the work status and measurement results, and fully studying the suitability of reinforcement works to the excavation method and tunnel support system.

2) In order that the reinforcement works can be employed efficiently and effectively, it is essential that the tunnelling method is well suited to the reinforcement works selected. Tunnel design and construction methods should be optimized to meet the field conditions by thoroughly evaluating the various design conditions. Figure 3-1 shows the flows of surveys, design, and tunnel construction, summarized from the viewpoint of the reinforcement works.

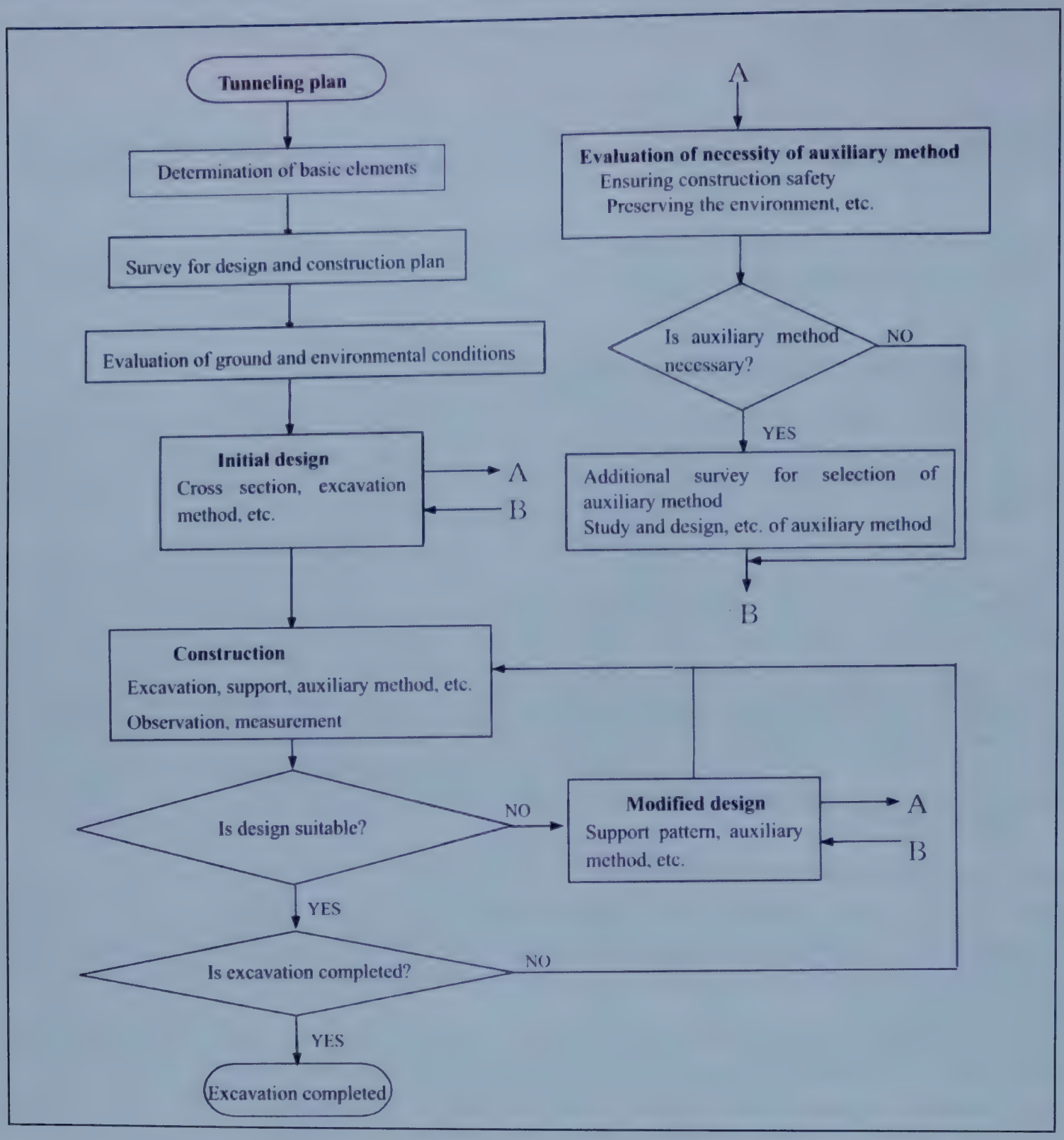


Figure 3-1: Flows of Surveys, Design, and Construction Focusing on Reinforcement works

3.1.2 Application of Reinforcement Works

Reinforcement works are classified into broad categories according to their objectives: face stabilization, groundwater measures, prevention measures against the surface settlement, and measures for structures in the vicinity. Reinforcement works should be selected that are superior in terms of safety, effects, and cost performance, evaluating the conditions ahead of the face, the face itself and behind the face, on the basis of comprehensive study, always involving the newest technology information. Table 3-1 lists typical techniques among various reinforcement works currently used.

Most reinforcement works have both major and secondary effects. Several reinforcement works are often simultaneously used. Efficient and cost-effective reinforcement works should be selected by thoroughly evaluating not only tunnelling methods and rock conditions, but also environmental conditions.

At the time of initial design, available survey results are limited. It is, therefore, possible to estimate the range where some reinforcement works might be necessary, but it is difficult to specifically determine the extract range. Accordingly, in the

construction stage, regardless of the range assumed in the initial design, it is necessary to establish the suitable range to which reinforcement works are to be applied, referring to the face observation and monitoring results, while considering work safety, cost efficiency and effectiveness of the work.

Table 3-1: Classification of Reinforcement Works

Method			Purpose							Applicable Earth			Remarks
			Work Safety				Environmental Preservation						
			Tunnel stabilization										
			Roof	Face	Sidewall	Groundwater Control	Groundwater Control	Ground surface settlement control	Protection of structures in the vicinity	Hard rock	Soft rock	Soil	
Roof Support	Forepoling		+							+	+	+	
	Steel pipe forepiling		+					+	+		+	+	*2
	Pipe roof		+					+	+		+	+	*1
	Horizontal jet grouting		+	+	+			+	+			+	*1
	Slit concrete		+					+	+		+	+	*1
Face Support	Face shotcrete			+						+	+	+	
	Face bolt			+						+	+	+	
	Long face bolt			+				+		+	+	+	
Wall Foot Support	Foot bolt				+						+	+	
	Foot pile				+						+	+	
	Temporary invert				+						+	+	
Groundwater Control	drainage	Drainage boring	+	+	+	+				+	+	+	
		Well point	+	+	+	+						+	
		Deep well	+	+	+	+						+	
		Drainage drift	+	+	+	+				+	+	+	
	cut-off	Grouting	+	+	+		+	+	+	+	+	+	
		Cut-off wall					+	+	+			+	
Ground Improve	Grouting		+		+			+	+			+	
	Vertical piles		+		+			+				+	

+: Standard measure
**1: Measures that are difficult to implement with conventional tunnel construction machinery and facilities control.*
**2: Method that requires modification of tunnel construction machines and materials spending upon the specific technique selected.*

(1) Tunnel Face stabilization

The mountain tunnelling method is predicated on the stability of the tunnel face and the tunnel crown until supports have been built. In cases where the tunnel face is unable to support itself until the completion of the support, it is necessary to take appropriate face stabilization measures to ensure safe and efficient construction.

When face stability cannot be maintained, the excavation section may be divided or a core may be left. However, it would be more realistic in some cases to excavate a large section, not divide it into small portions. Locations conditions and work efficiency, appropriate reinforcement works should be planned based on a thorough evaluating of rock conditions. In the study for selection of face stabilizing measures and combination of several methods, the necessity of groundwater measures should be the primary concern, since face stability is noticeably affected by water inflow through the face of other sections.

Next, reinforcement works should be properly selected to stabilize different sections, i.e., crown, face, and foot. It is also important to plan in advance proper measures

anticipating problems with face instability, and based on visual inspection and monitoring results, to take suitable measures as soon as possible.

(2) Control of water inflow

When water flows or is anticipated to flow into the tunnel through the tunnel face, groundwater needs to be drained to stabilize the face and improve construction efficiency. However, caution should be exercised if water inflow is to be controlled because it could result in surface settlement.

if drainage in the tunnel could cause depression of the groundwater and the ground surface settlement, the control of water inflow is not a practical solution. The commonest method applicable in such a case is grouting. Grouting is most reliable for stabilizing the face, which will reduce water inflow volume and improve the rocks. This method is therefore used when there is a large volume of water inflow and in discontinuous layers with sand intercalations where water drainage or weep holes are not effective. Grouting is also used in urban areas when lowering the groundwater table is not practical, or surface settlement needs to be controlled by improving the rocks.

Other methods such as freezing and bulkhead are also used in some cases.

(3) Control of surface settlement

Since surface settlement during tunnelling is closely related to topographical and geological conditions, groundwater, construction methods, and other factors, it is difficult to specifically determine the cause. However, major causes of the surface settlement are the loosening of rocks induced by tunnel excavation and the removal of groundwater.

Methods for controlling surface settlement during tunnel construction include steel pipe forepiling, pipe roofing, horizontal jet grouting, and slit concrete method. When surface settlement due to groundwater drainage must be prevented, grouting methods which use chemicals such as suspension-type water glass or a solution-type water glass may be used.

(4) Protection of structures in the vicinity

When tunnels are excavated in urban areas, sometimes there are cases where tunnelling in the vicinity of existing surface structures, such as buildings and bridges in unavoidable. In such cases, measures are taken as needed to protect those structures.

In case that a tunnel is excavated directly below or in the vicinity of the road, railway track, water channel, building, or underground structure, the allowable limit or control criterion should be set up through preliminary discussion with the managers of the structures in the vicinity, and proper measures should be taken to protect the existing structures

3.2 Reinforcement Works for Safety of Tunnelling

3.2.1 Reinforcement Works for Face Stabilization

Generally, the term 'face stability' refers to the stability of the cutting face zone (including the face itself). Tunnel face stabilization works can be classified into three categories according to the site at which they are applied: (1) stabilization of the crown, (2) stabilization of the face, and (3) stabilization of the footing. For face stabilization, it is important to plan countermeasures against possible instability in advance, to continually evaluate the face stability by face observations, etc., and to execute the plan before it is too late.

(1) Stabilization Works for Crown

i) Filing type forepoling
Filing type forepoling is a reinforcement works where bolts, steel bars, or pipes with a length of less than 5m, are driven into rocks around the top heading arch, as shown in Figure 3-2. The aiming method is to increase the apparent shear strength of the rocks at the crown and to prevent the loosening of the rocks ahead of the face. Generally, steel reinforcing bars are used, with the space around bars usually being filled with cement paste or mortar. Occasionally the space is not filled depending on rock conditions at the face.
This method is generally used for preventing the collapse or falling of the crown and is often implemented at an early stage of stabilization.

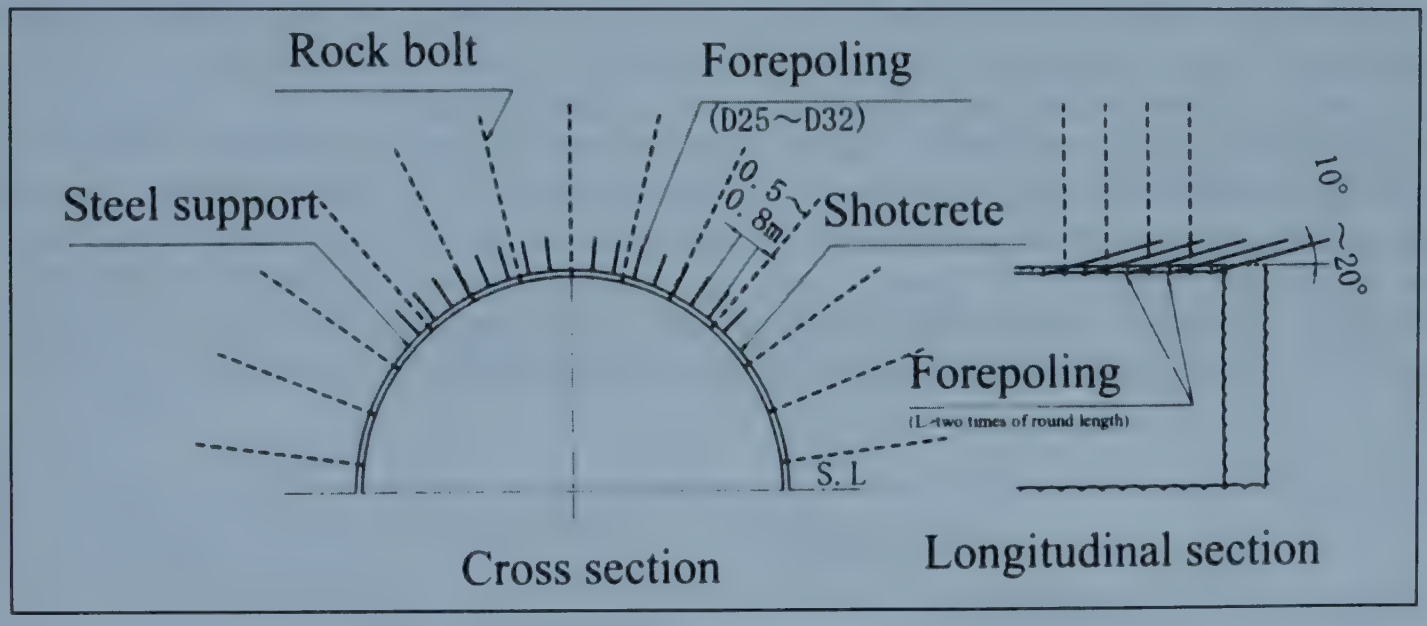


Figure 3-2: Example of Filling Type Forepoling

ii) Grouting type forepoling
Grouting type forepoling is a reinforcement works where bolts or pipes, with a length of less than 5m, are driven diagonally into the rocks ahead of the face, with simultaneous injection (under pressure) of quick-setting cement paste or chemical grout, in order to enhance the stability of the crown ahead of the face, as shown in Figure 3-3.

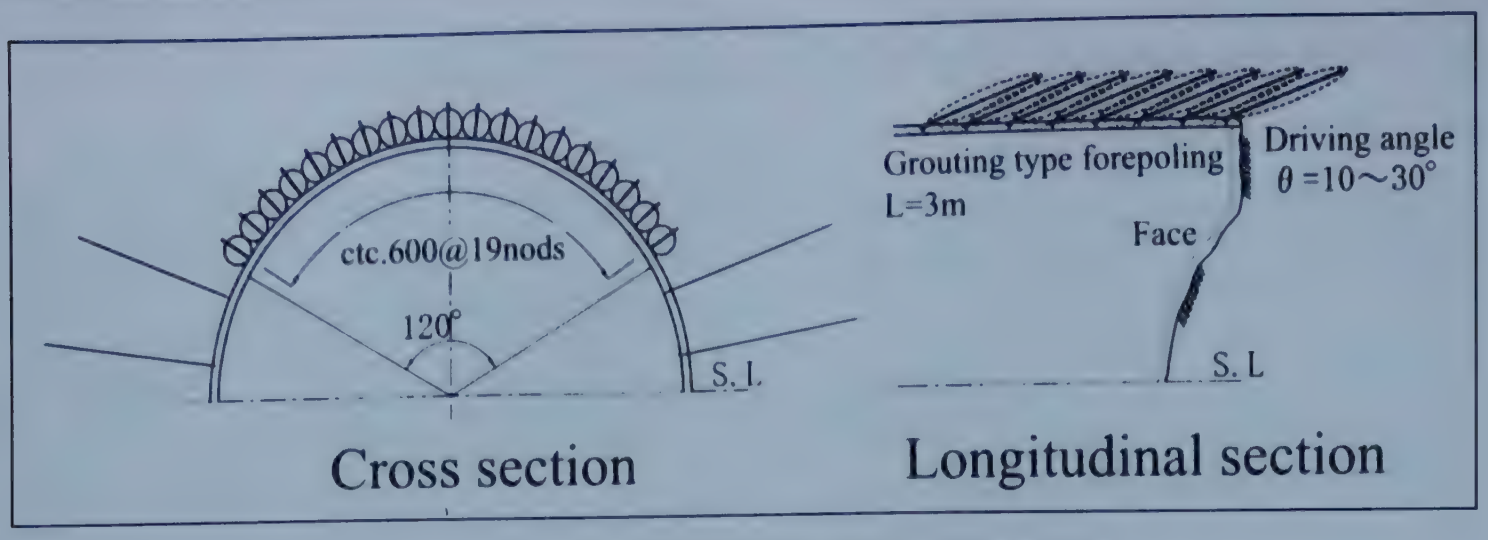


Figure 3-3: Example of Grouting Type Forepoling

iii) Pipe forepiling

Pipe forepiling is one of the reinforcement works for reinforcing unstable rocks, such as talus, fault fracture zones, or soil, where the ground arch action is not expected. An additional benefit of this method is a reduction in prior displacement. Usually, steel pipes longer than 5m in length are used. This method is mainly used for the stabilization of the crown. However, it may require a change in the excavation cross-section or the kind of machinery used in the application. Therefore, it is necessary to check the time schedule before adopting this method.

Pipes are inserted into the rocks along the upper perimeter of the tunnel prior to tunnel excavation, but the exact arrangement of pipes varies depending on the rock conditions and the position of nearby buildings. There are two construction methods; one uses specialized machines and the other uses a drill jumbo. The latter method can be applied in two ways: (a) a widening type in which the tunnel diameter is enlarged (to achieve a small driving angle for steel pipes); or (b) a non-widening type.

Filling type forepiling, which can also be used for improving rock stabilization, achieves tight contact between steel pipes and rocks by filling the void space between the steel pipes and the rocks with a grout, such as cement paste. Another example is the grouting type forepiling method that forms composite reinforcements consisting of steel pipes and grout materials, where the rocks are reinforced by injecting cement paste or chemical grout into the rocks around the steel pipes (see Figure 3-4)

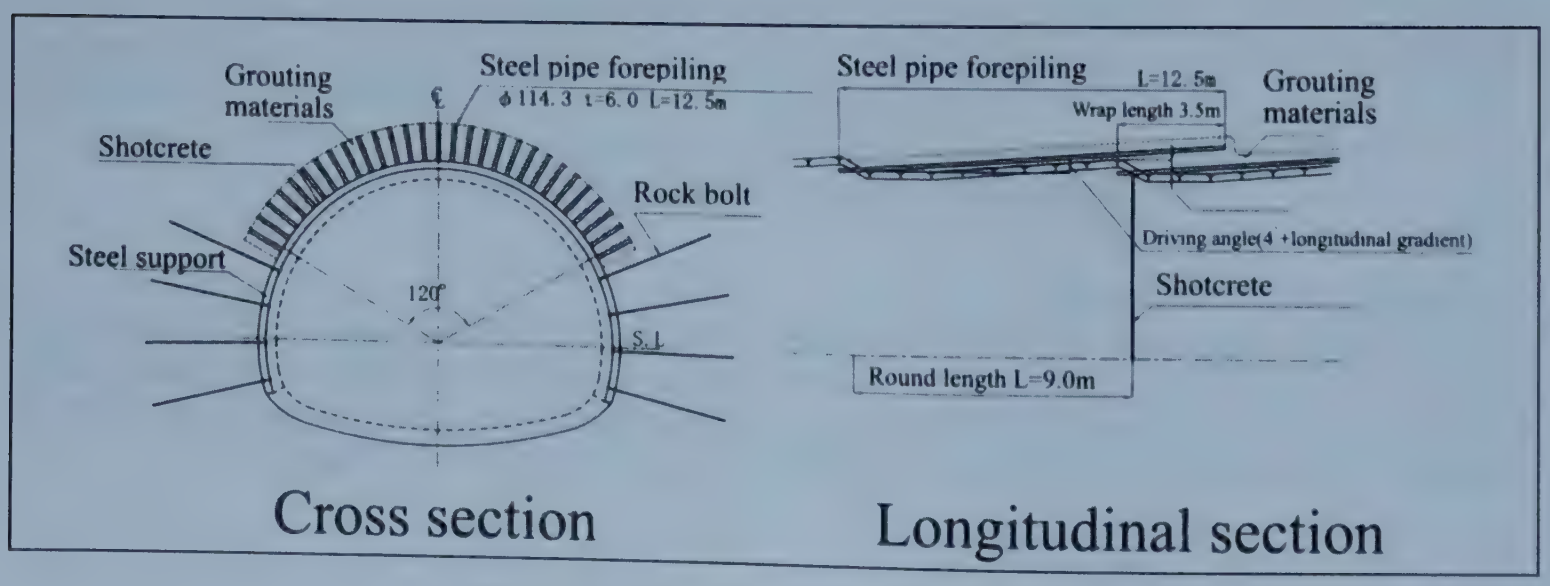


Figure 3-4: Example of Pipe Forepiling

(2) Stabilization Works for Tunnel Face

There are rock stabilization methods for the tunnel face such as ring cutting, shotcreting, face bolting, or grouting. face stability (i.e. stand-up time) is affected by both scale and

shape of the tunnel face. Since the purpose of excavating a small section is to improve the face stability, it is important for face stabilization to spray concrete as early as possible, by excavating the face part by part. On the other hand, it may be reasonable to excavate the large tunnel in conjunction with reinforcement works such as face shotcrete or face bolt. In order to make this decision, it is necessary to examine the rock conditions, location conditions, or practical workability in a comprehensive manner.

i) Face shotcrete

Face shotcrete is sprayed onto the face at the thickness of 3 to 10cm as soon as it is excavated in order to increase the stand-up time of the face, as shown in Figure 3-5.

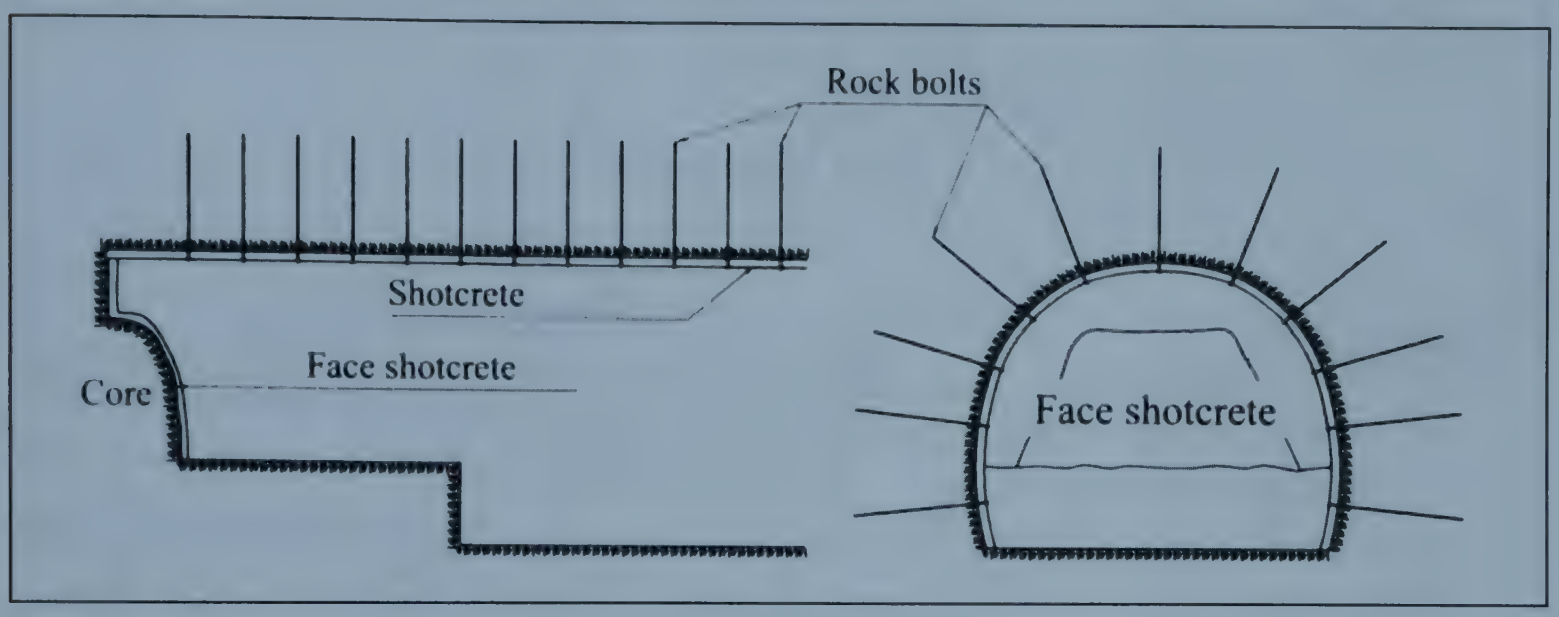


Figure 3-5: Example of Face Shotcrete

ii) Face bolt and long face bolt

There are two types of face bolts. One is slightly shorter than 5m in length, the other is longer than 5m. The purpose of the face bolt is to stabilize the tunnel face and reduce the ground surface settlement by supporting a part or the whole of the face with rock bolts. Face bolting is more effective when used in conjunction with face shotcreting. The length of the face bolts should ideally maintain an effective residual length when they are cut by tunnel excavation.

Grouted face bolts may be used for an increased reinforcement effect. Glass fibre reinforced plastic bolts are often used because they are easy to cut.

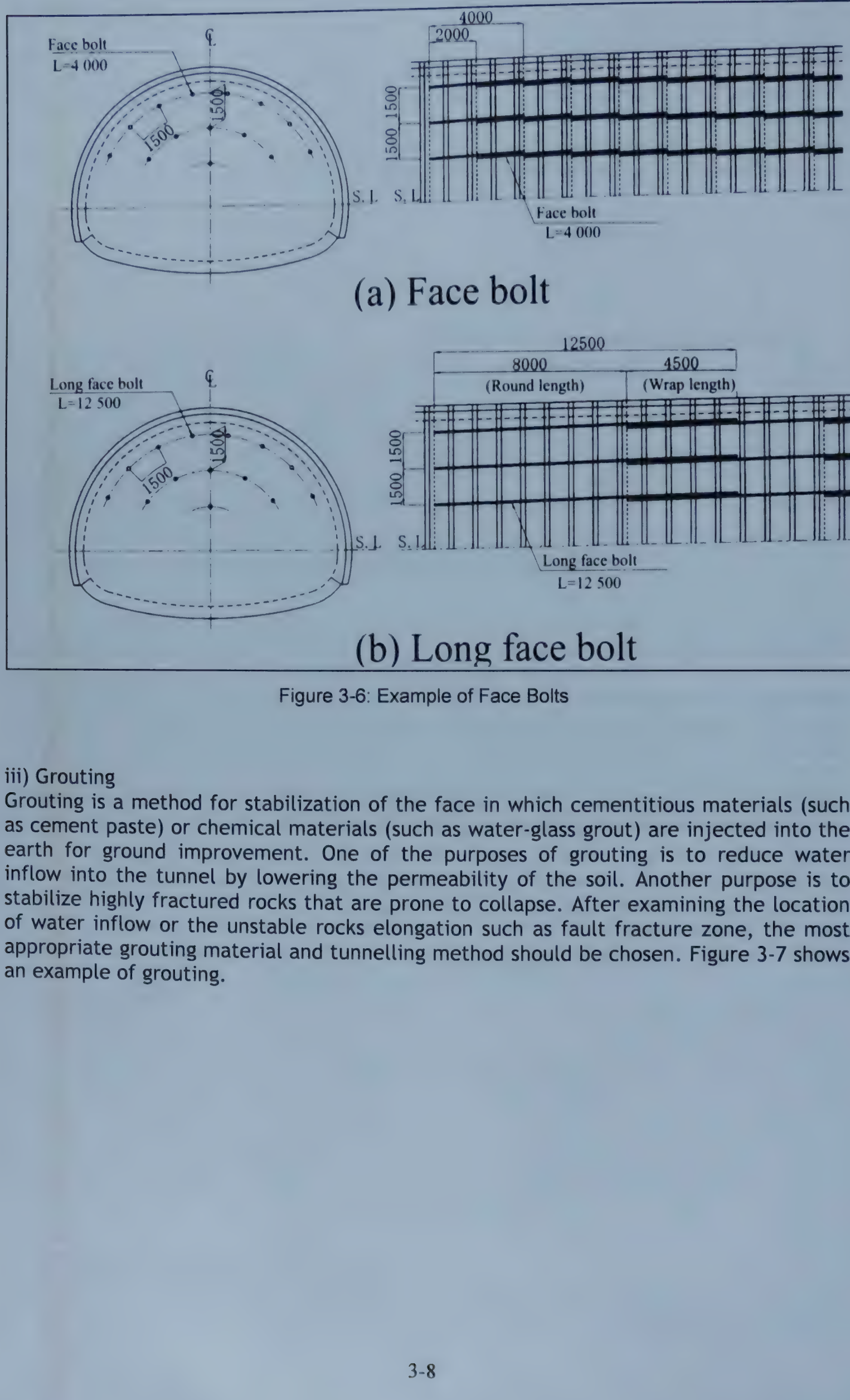


Figure 3-6: Example of Face Bolts

iii) Grouting

Grouting is a method for stabilization of the face in which cementitious materials (such as cement paste) or chemical materials (such as water-glass grout) are injected into the earth for ground improvement. One of the purposes of grouting is to reduce water inflow into the tunnel by lowering the permeability of the soil. Another purpose is to stabilize highly fractured rocks that are prone to collapse. After examining the location of water inflow or the unstable rocks elongation such as fault fracture zone, the most appropriate grouting material and tunnelling method should be chosen. Figure 3-7 shows an example of grouting.

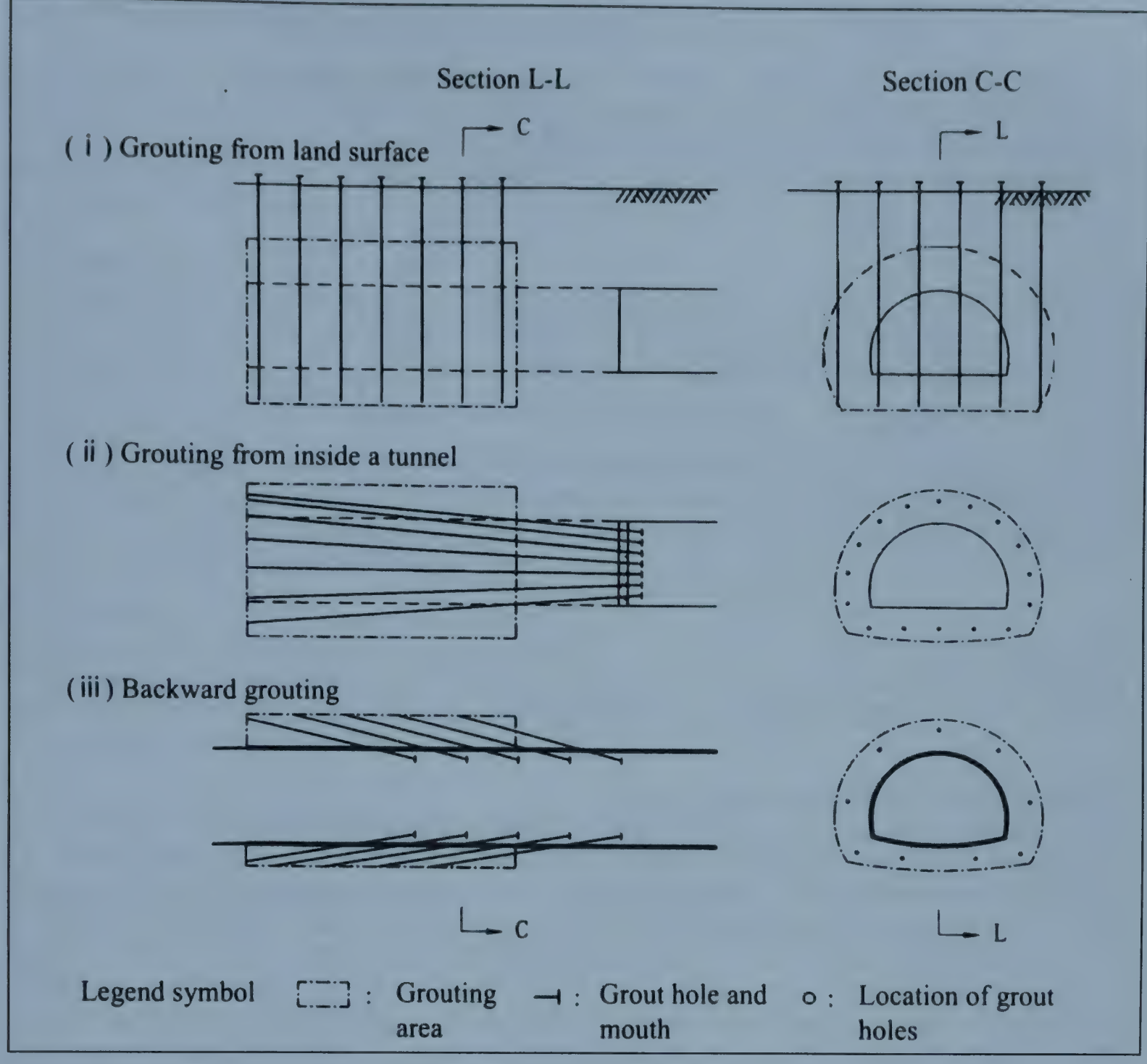


Figure 3-7: Example of Grouting

(3) Stabilization Works for Footing

Stabilization methods for the footing help protect against damage caused by both foot settlement and the associated rock loosening (caused by the bearing capacity of ground at the footing of supports being insufficient). Methods include shotcreting (for a top heading temporary inverted arch), downward bolting (in which rock bolts or steel pipes are used at the footing or supports), or jet grouting. These methods increase the bearing capacity of the ground.

i) Temporary invert in the top heading

A top heading temporary invert is temporary closed the top heading by means of shotcrete. This is a very efficient method, one that has the advantage of being able to proceed with the works in accordance with the face conditions and the monitoring results. However, it reduces the ease of excavation of the top heading and decreases work efficiency on the bench by means of demolishing the invert. Since a large deformation may be induced when demolishing the invert, the application of the method needs careful prior examination (see Figure 3-8).

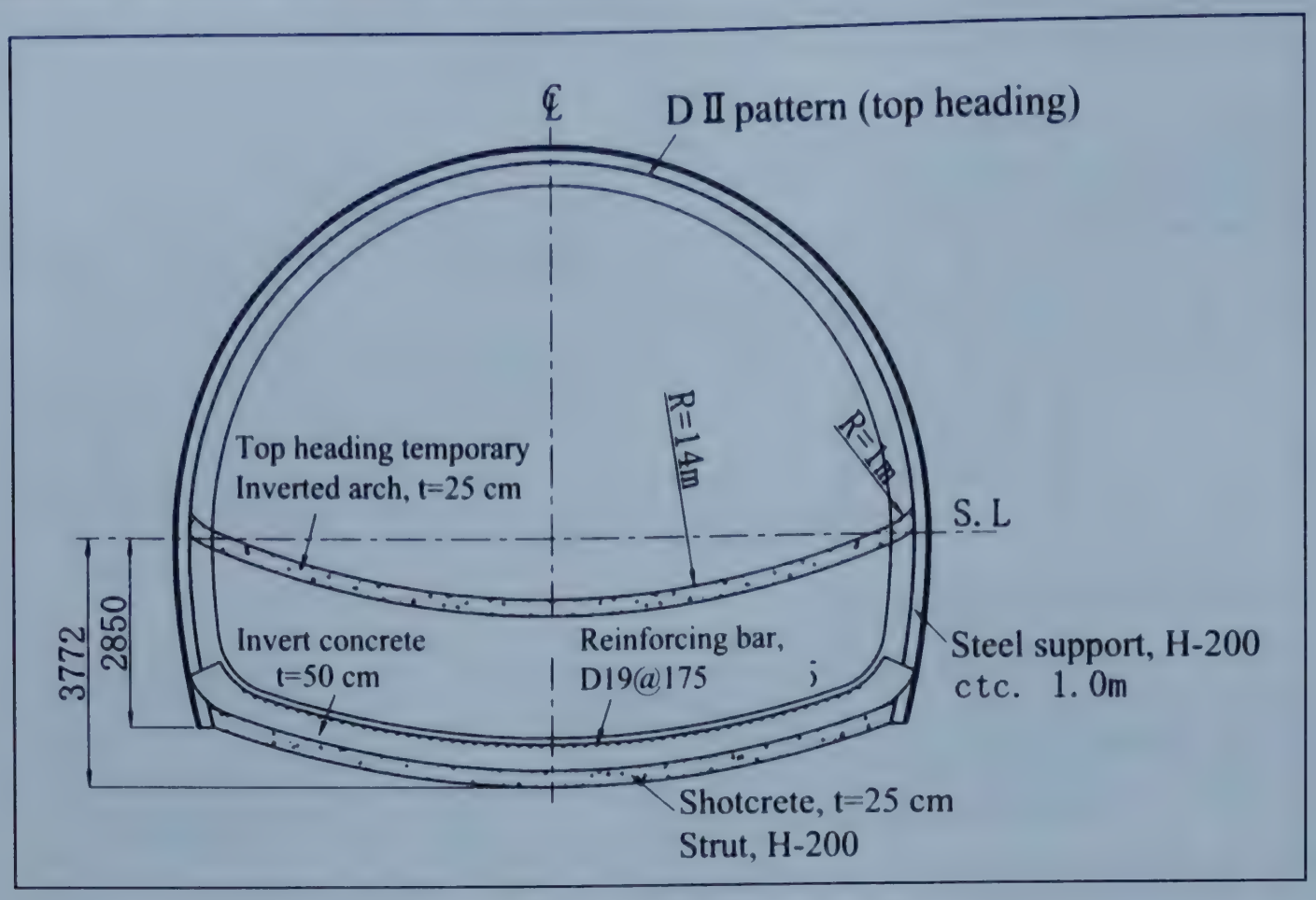


Figure 3-8: Example of Top Heading Temporary Invented Arch

ii) Foot reinforcement bolting and piling

Foot reinforcement bolting and piling (installation of downward-facing rock bolts at the footing of supports or small-diameter steel pipes, and jet grouting) are aimed at reducing the stresses in the contact rocks of the top heading and preventing the collapse of the rocks during bench excavation. If the strength of the rocks around the footing is too low, then injecting bolts and simultaneously grouting under pressure with cement paste or chemical grout can increase the strength of the rocks. The method can have the opposite effect if the drilling water disturbs the rock while injecting bolts or small-diameter steel pipes. Therefore, a careful selection of drilling methods is required. Figure 3-9 shows an example of foot reinforcement piles for purposes of reducing tunnel deformation and achieving the required base rock bearing capacity.

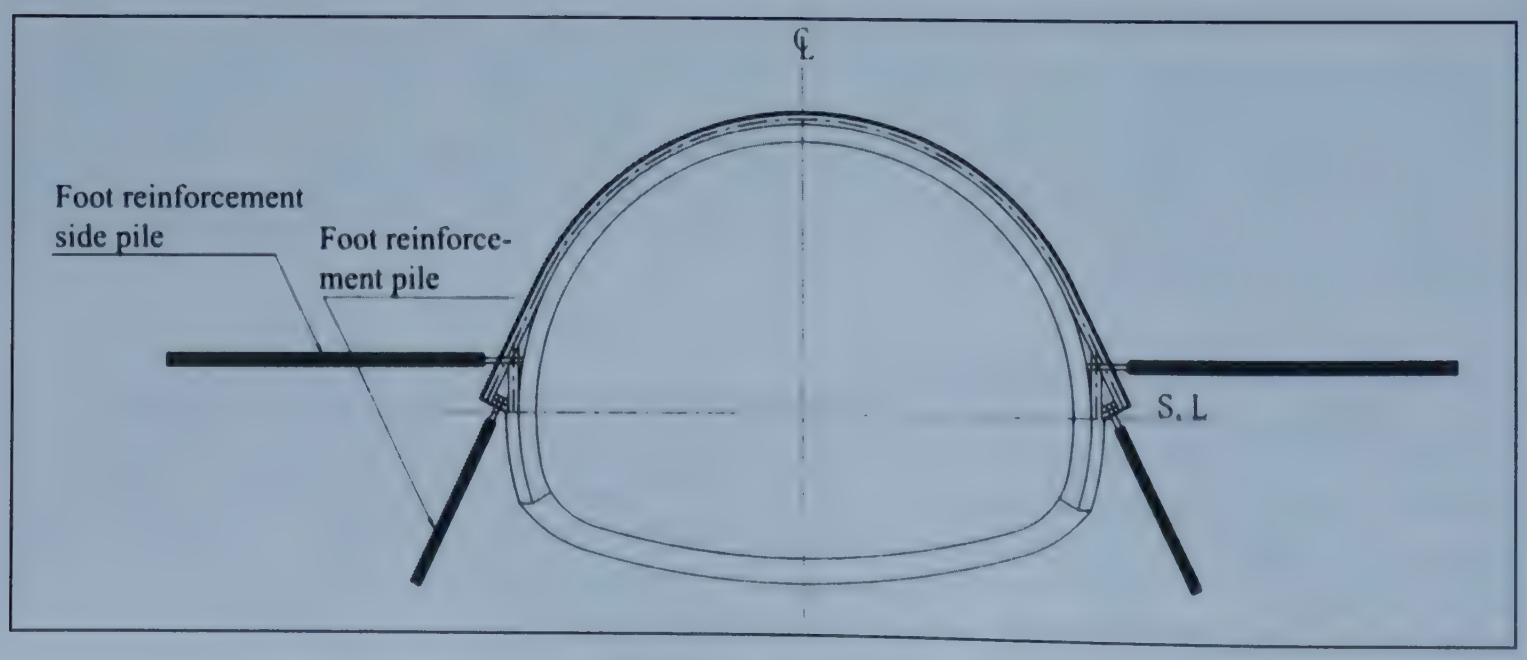


Figure 3-9: Example of Foot Piles

3.2.2 Reinforcement Works for Controlling Water Inflow

If the rate of water inflow in tunnel excavation is high, it will cause problems including lowering of the face stability (resulting in difficulty in tunnel excavation), the collapse of the face, insufficient adherence of shotcrete and rock bolts and decreased tunnelling work efficiency. Control works of water inflow can be classified into two categories: drainage and water sealing.

(1) Drainage

Drainage is more widely used than water sealing methods. Since there can be cases in which lowering of the groundwater table may be inadequate (due to ground surface conditions in the surrounding environments, and/or the abundance of groundwater), drainage methods should be selected with due consideration.

i) Drainage boring

Drainage boring is a widely used method in which water is removed through boreholes drilled by a boring machine or drill jumbo in order to lower water pressure and groundwater table. Careful drainage boring works in soil should be taken not to remove soil particles with the water.

ii) Drainage drift

Drainage drifts, an approach of providing a pilot drift of small diameter, are often used in conjunction with a drainage boring, for sites of large amount groundwater. When an aquifer with a good amount of high water pressure inflow is widespread, many drainage drifts may be required.

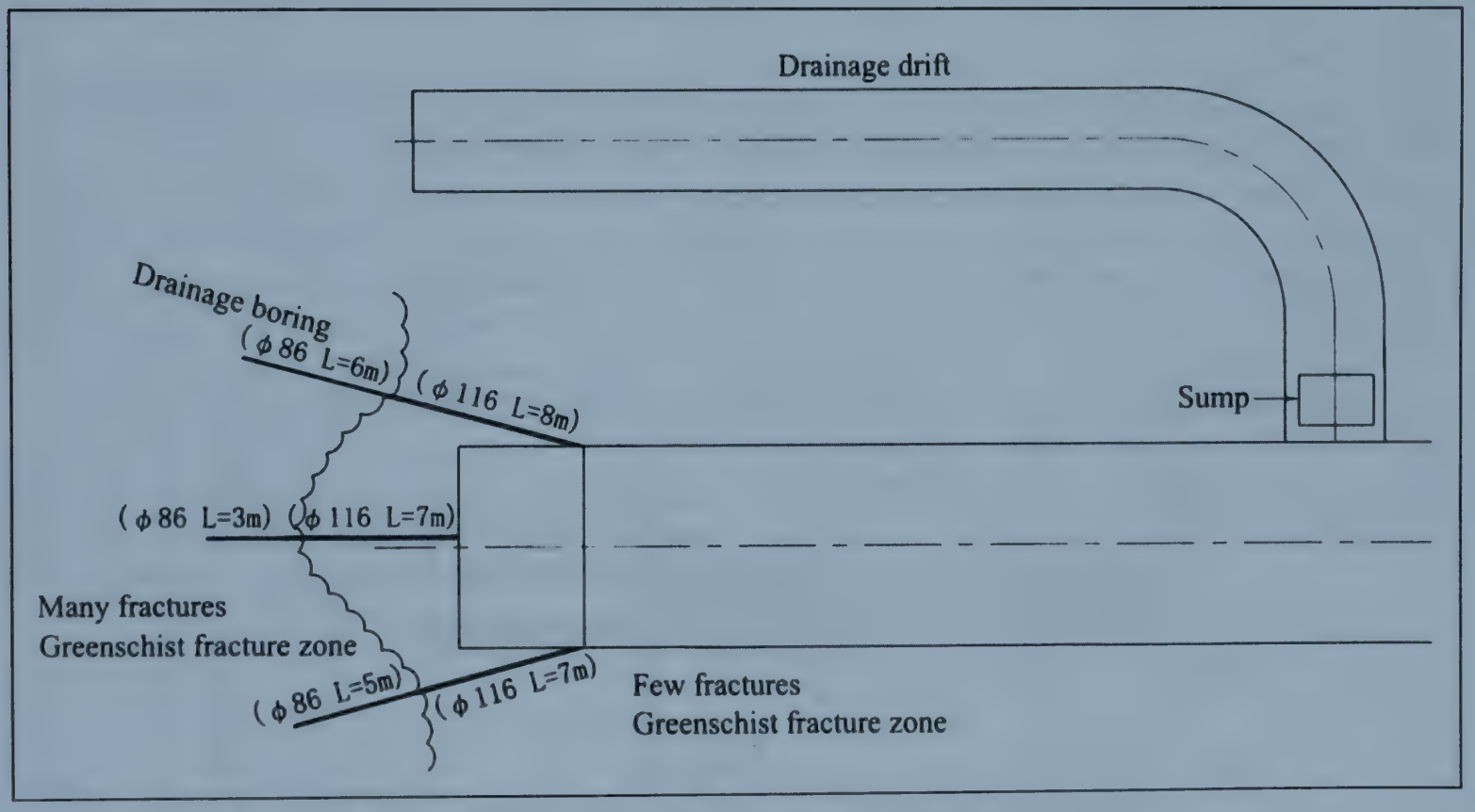


Figure 3-10: Example of Combined Use of Drainage Drift and Drainage Boring

iii) Well point drainage

Well point drainage is a method used for removing groundwater under negative pressure, in which groundwater collecting pipes called 'well points' are inserted into the ground.

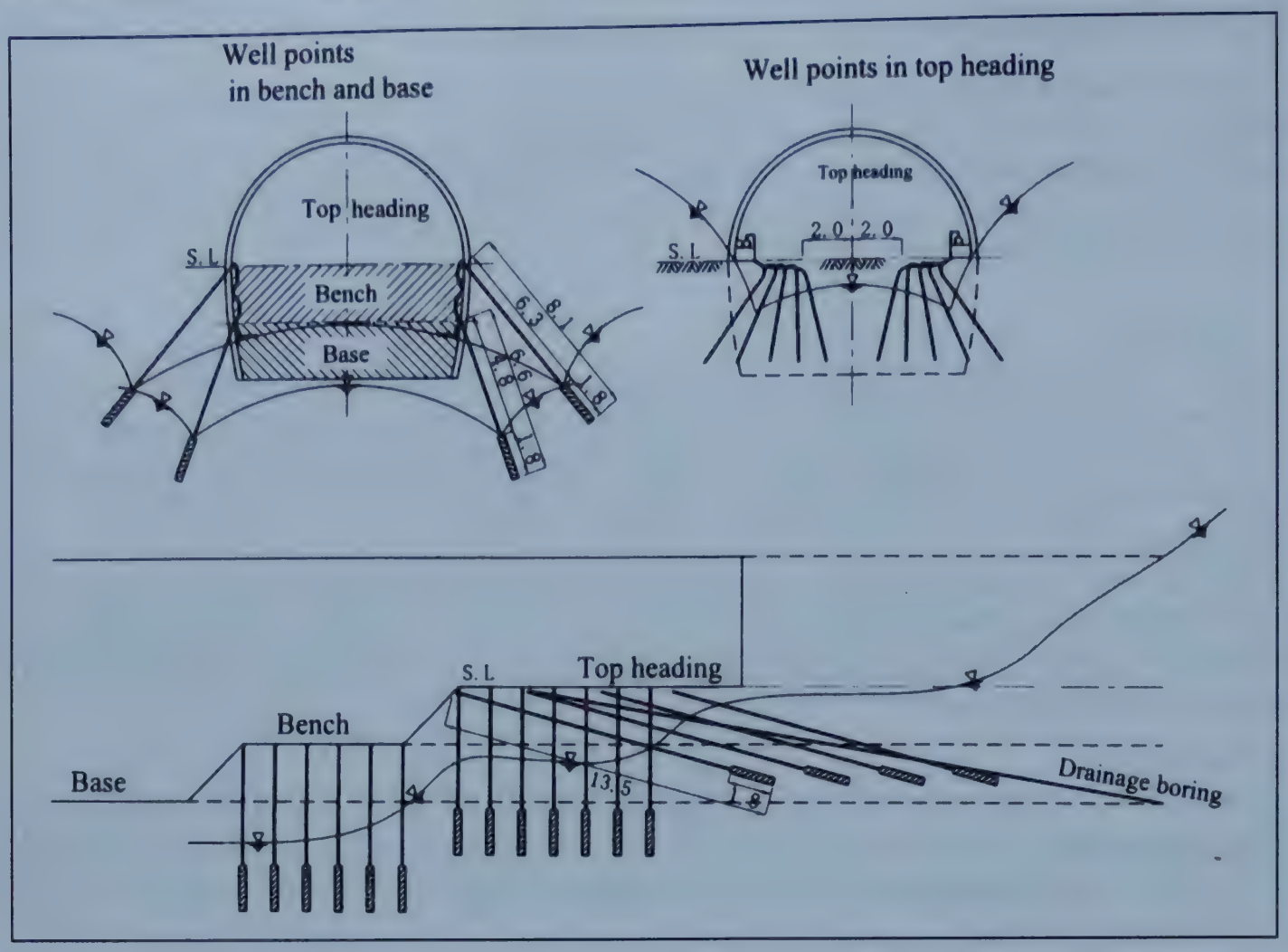


Figure 3-11: Example of Well Point Sunk from Inside the Muginda Tunnel

iv) Deep well

The deep well is a drainage method in which a deep well, usually about 300 to 600m in diameter, is sunk and a submersible pump is used to remove water.

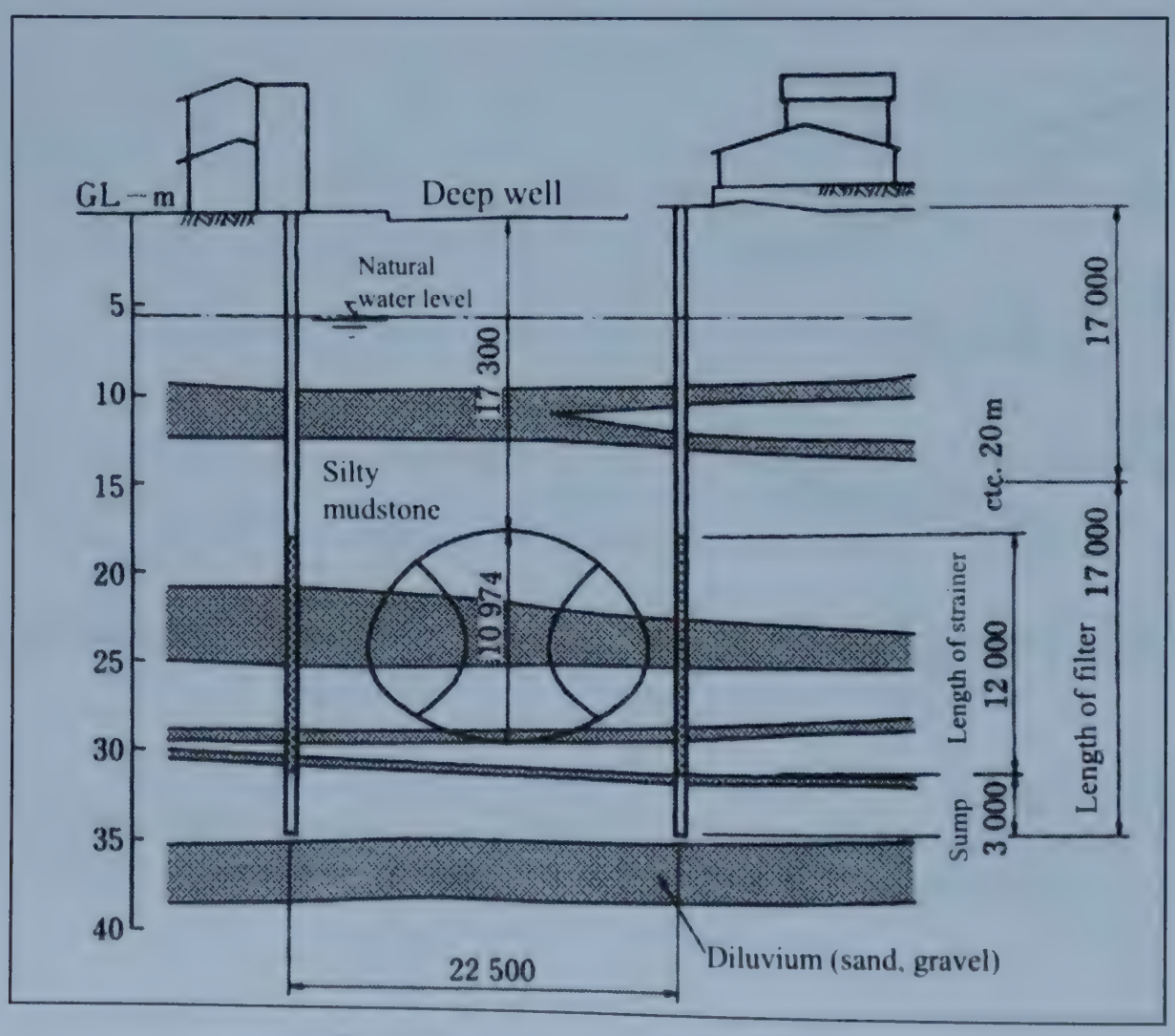


Figure 3-12: Example of Deep Well

(2) Water Sealing

Although drainage is usually used to control water inflow, there are certain circumstances where water sealing is useful instead of (or in addition to) drainage. Examples include situations where: the amount of water inflow cannot be reduced by drainage, the dewatering method cannot be used due to the risk of ground surface settlement, or where external environmental considerations do not allow groundwater lowering or depletion. However, even in these cases, water-sealing methods are more expensive than drainage methods and caution should be exercised regarding the surrounding environment.

There are a number of methods for water sealing including grouting methods, cut-off wall methods. The grouting method aims to achieve sealing by reducing the permeability of the ground. Cementinuous materials (like cement paste) or chemical grouting materials (like glass water grout) are injected into the ground ahead or around of face in order to close the bleeding channel such as void or fissure. This method is effective for both the reduction of water inflow and the face stabilization (due to its ground improving effect). In mountainous areas, the grouting method is often used in conjunction with drainage methods, since drainage methods alone are insufficient to control water inflow. Refer to Reinforcement Works for Existing Structures for the cut-off wall method.

In general, chemical grout is more permeable than cement-type grout and solidifies in a shorter time. Therefore, chemical grout is more suitable for controlling water inflow. In using the grouting method, suitable materials and grouting method should be determined by pre-test grouting and based on the grouting area, the ground conditions, the water pressure, the amount of water inflow, etc. There are two injection methods. The first is injected from the land surface and the second is injected from inside the tunnel. The former has the advantage of not disrupting tunnel excavation. However, it is not economical when the overburden is thick. The application of this method requires careful consideration of conditions of performance, the construction schedule, and the cost. In the latter method, the bulkhead or temporary lining should be built in order to achieve good grouting. It has to be built for preventing the failure of the cutting face and the grout leakage in addition to checking the impact on the construction schedule. Careful control of grout pressure should be paid - too low a pressure and the grouting will be poor, too high and costs will increase (due to higher volume of grout used) and there may also be unwanted impact on the surrounding ground and/or buildings. The grout pressure has to be controlled in consideration of the geological conditions, the groundwater pressure, and the effect on structures in the vicinity. At the end of grouting, the change of the amount of water inflow, the injection volume, the grouting pressure, and the rock conditions in each borehole should be reviewed, and the performance of grouting should be checked by test borings (Figure 3-13).

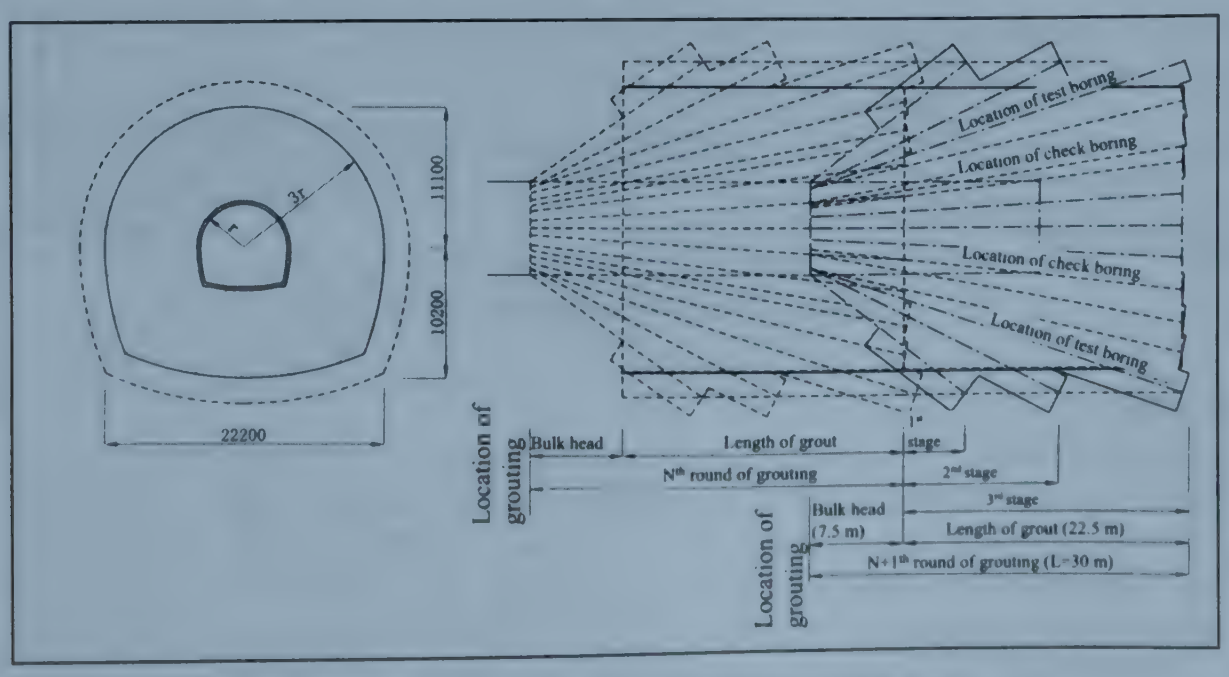


Figure 3-13: Example of Grouting from Inside a Tunnel

3.3 Reinforcement Works for Preservation of Surrounding Environment

3.3.1 Ground Surface Settlement

Reinforcement works such as pipe forepiling, pipe roof, horizontal jet grouting, slit concrete and vertical pre-reinforcement are used for preventing the ground surface settlement caused by the loosening of the ground during tunnel excavation. To prevent ground surface settlement caused by dewatering, prevention measures such as grouting or freezing are used. However, prevention measures for ground surface settlement are often used in conjunction with reinforcement works for face stability on the condition that the tunnel face will be stabilized. Since the imposed load on supports may often increase when the displacement of the rocks ahead of the tunnel face is restrained by pre-supports, reinforcement works for the improvement of bearing capacity of rocks or sizing up supports should be taken if necessary. Early enclosure by invert is very efficient for reducing the ground surface settlement. Also, the excavation sequence or the arrangement of division of heading should be carefully studied.

(1) Pipe Forepiling

Pipe forepiling prevents ground surface settlement by using steel pipes to reinforce the rocks by means of pipe stiffness and soil improvement. Long steel pipes about 50 to 125mm in diameter are placed around the circumference of the tunnel and grouting materials are injected through the steel pipes and into the rocks. The length of the pipes is about 12m. There are two methods of forepiling. The first uses a specialized machine, which can manage pipes of 12m in length as a single unit. The second uses a drill jumbo, where short pipes are connected together to reach the desired length. The former is often used for a long target zone, in which many houses are located immediately above the tunnel. This method can be applied in two ways; (a) a widening type in which tunnel diameter is enlarged along the driving angle of steel pipes; or (b) a non-widening type. Figure 3-14 shows an example of pipe forepiling carried out by a specialized machine in order to prevent the deformation of houses overlying twin tunnels.

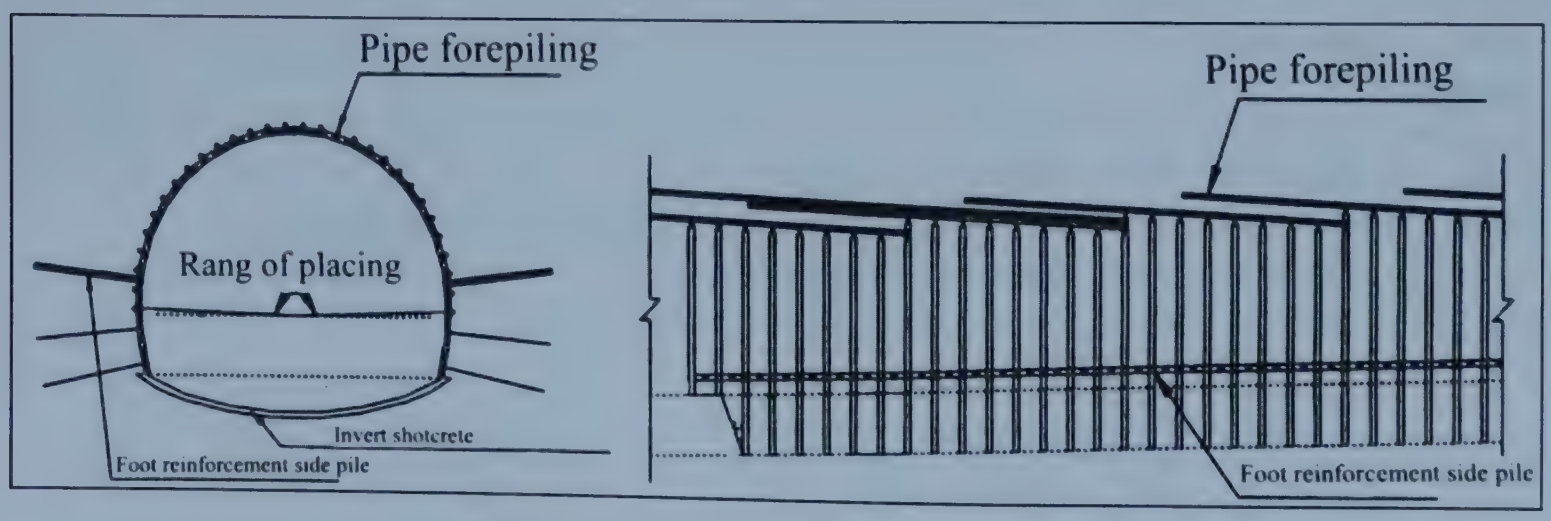


Figure 3-14: Example of Pipe Forepiling

(2) Pipe Roof

A pipe roof is a method that reduces the ground surface settlement by reinforcing the earth surrounding the tunnel with steel pipes. In this method, steel pipes are inserted while horizontally boring around the circumference of the tunnel, and the voids inside and outside the pipes are grouted. In many cases, this method is used at portals not allowing enlargement, because on the circumference of the tunnel bore, steel piles are driven in parallel with the tunnel axis for the protection of structures in the vicinity or

when boring a tunnel with shallow overburden. However, this method needs a relatively large temporary facility such as a reaction wall and the wok speed is relatively slow. Therefore, work conditions and schedules should be taken into account when selecting this method.

Figure 3-15 shows an example of a large-diameter pipe roof used for preventing the surface subsidence at a heavily travelled main road which is located above the tunnel.

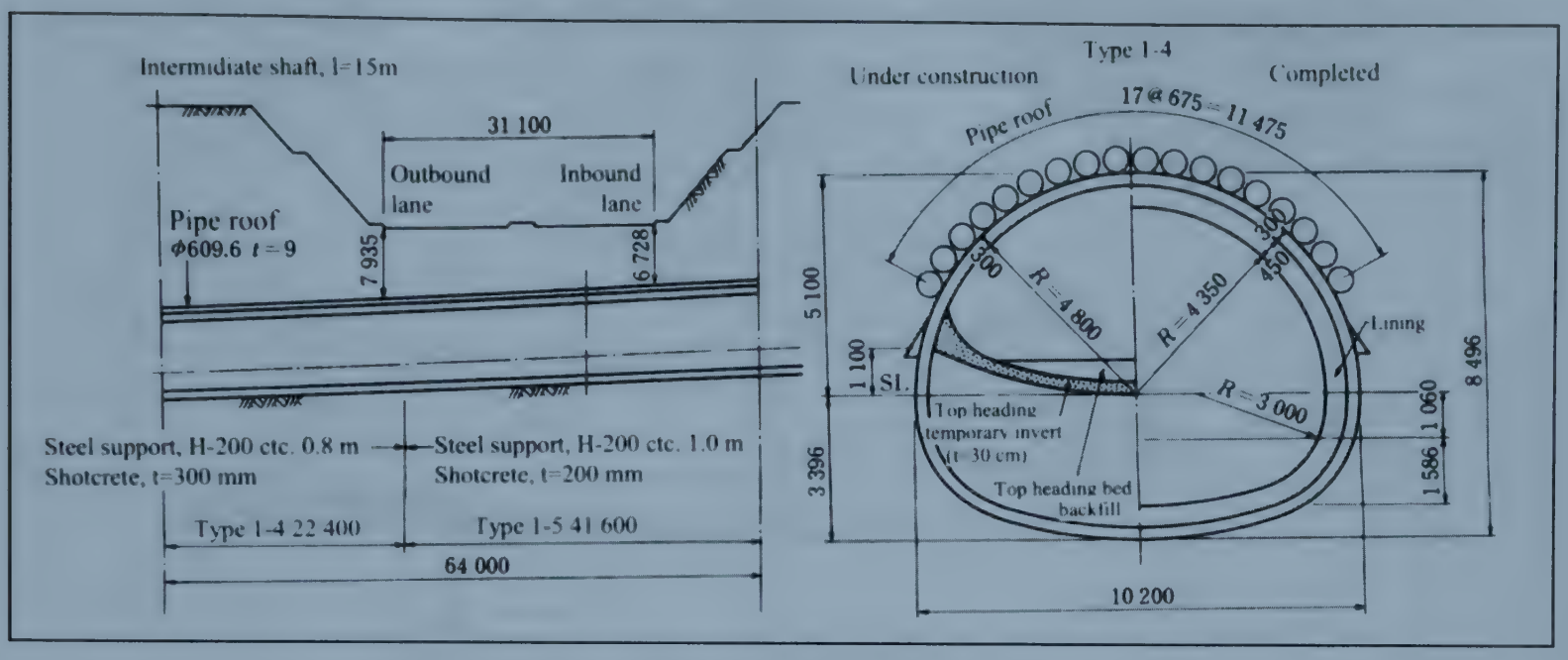
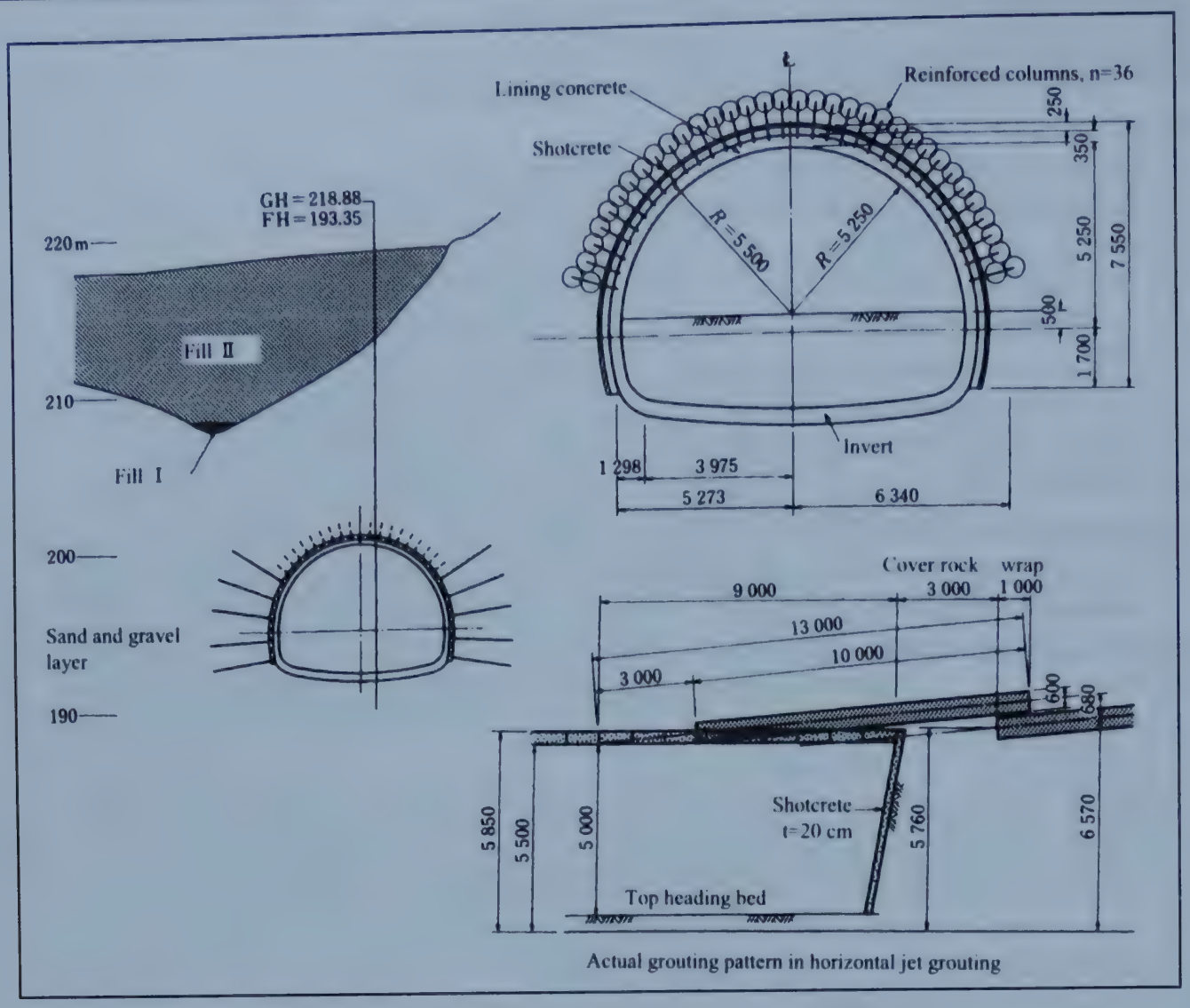


Figure 3-15: Example of Large-Diameter Pipe roof

(3) Horizontal Jet Grouting

In this method, an arch-shell-like reinforced zone is formed in the rocks ahead of the face in order to reduce the ground surface settlement. Generally, grout holes 10 to 15m in depth are drilled with a specialized machine. As the rotating drill rod is gradually extracted, high-pressure jets of cement-type solidification materials are injected from the end of the drill rod, creating a column made from a mix of soil and solidification material is formed. The reinforced zone can be formed by placing the required number of such columns along the circumference of the tunnel. There is a variant of this method that installs steel pipes in the columns in order to enhance longitudinal stiffness. Figure 3-16 shows an example of horizontal jet grouting used for a shallow tunnel.



Actual grouting pattern in horizontal jet grouting

Figure 3-16: Example of Horizontal Jet Grouting

(4) Slit Concrete

This method constitutes an approach of cutting an arched ditch, 3 to 5m long and 15 to 50cm thick, which is provided in the rocks ahead of the face along the circumference of the tunnel by a specialized machine. The ditch, immediately after cutting, is filled with concrete or other materials to construct the lining in a form of an arch shell. Because a continuous rigid structure along the transverse direction of the tunnel is formed, prevention of displacement ahead of the face can be expected. This method is suitable for tunnels in urban areas (where little surface settlement is allowable), or which pass close to important facilities.

Since the load caused by constraining the rocks ahead of the tunnel face will be increasing, Improvement of the bearing capacity at the foot of slit concrete and steel ribs should be considered. One cutting method uses a chain saw and another uses multiple auger screws. Figure 3-17 shows an example of slit concrete method used to enlarge the existing tunnels and Figure 3-18 shows an example of the slit concrete method used for preventing the ground surface settlement.

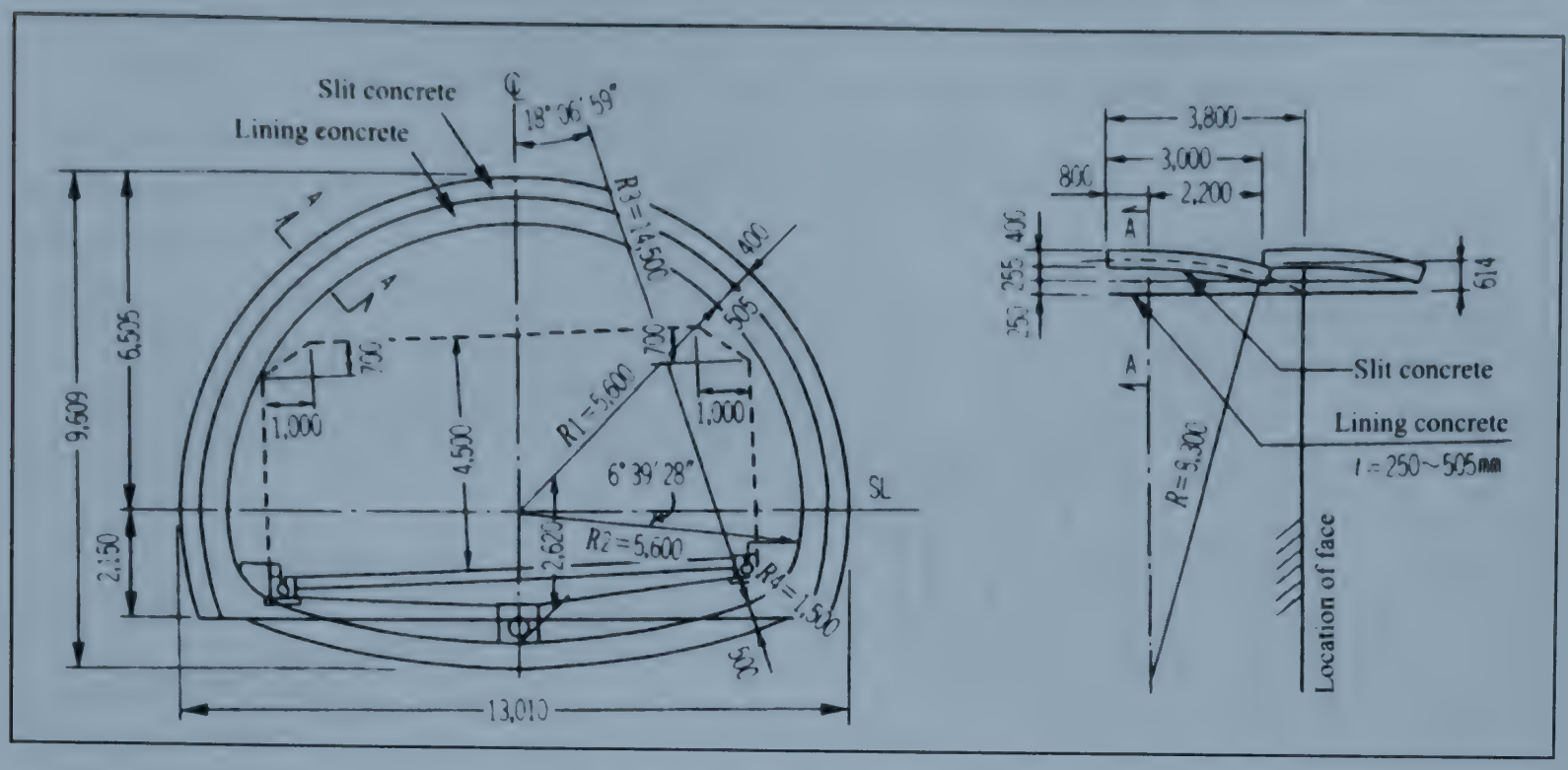


Figure 3-17: Example of Slit Concrete

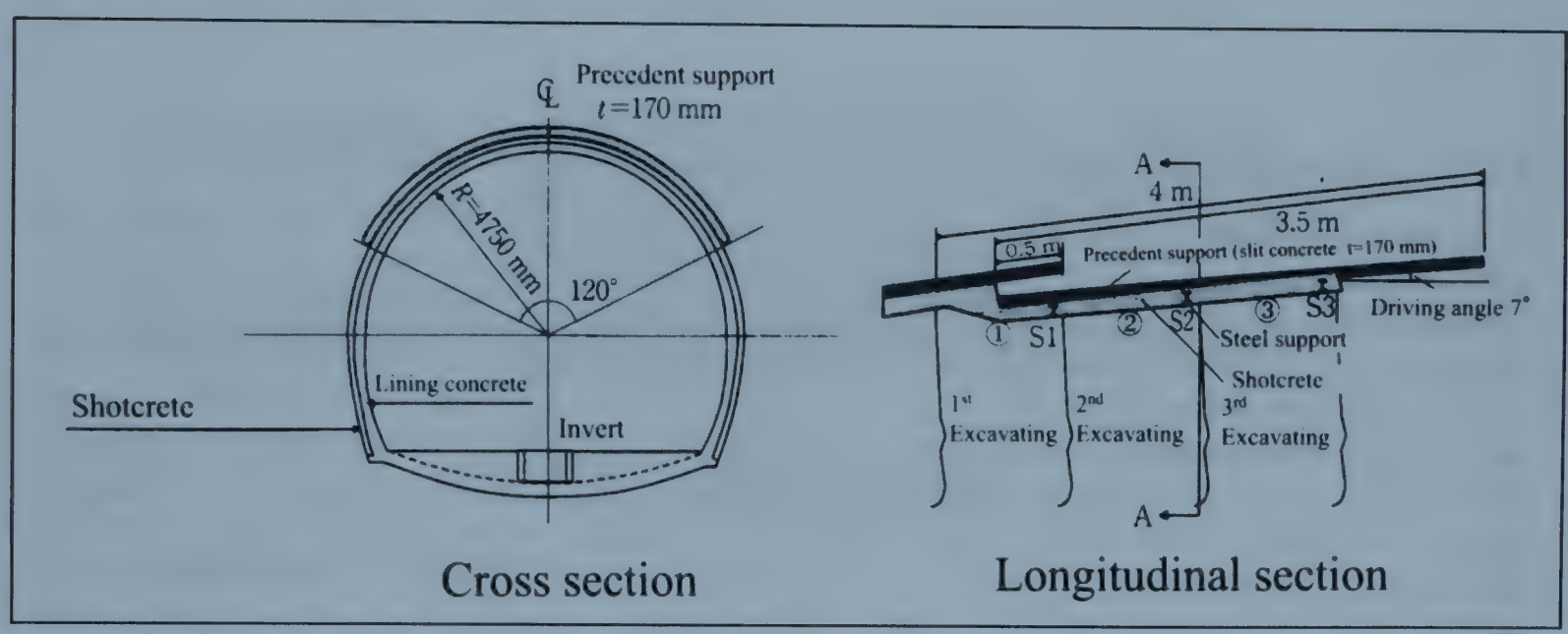


Figure 3-18: Example of Slit Concrete

(5) Vertical Pre-Reinforcement

Vertical pre-reinforcement is a method in which boreholes are drilled more or less vertically from the ground surface before excavating the tunnel, the reinforcing bars are inserted into these holes and fully bonded with filler, such as mortar. In this way, vertical pre-reinforcement is able to reinforce the ground and should be effective in reducing the settlement ahead of the face, or in preventing the loosening of ground from the beginning of excavation until rock bolt bonds have strengthened or shotcrete is applied. This method has the advantage that there is no need to compete with excavating work, but this method may be not allowed depending on circumstances of land use.

(6) Grouting

Grouting may be sometimes used for preventing ground surface settlement due to the effect of arch action caused by reinforcing the ground. It may also be used for preventing consolidation settlement of cohesive soil caused by dewatering methods used in tunnel excavation. The grouting material and the work method should be selected in

accordance with the purpose for which they are being used. Figure 3-19 shows an example of the combined use of vertical pre-reinforcement and chemical grouting in order to prevent ground surface settlement in soil located beneath houses and roads.

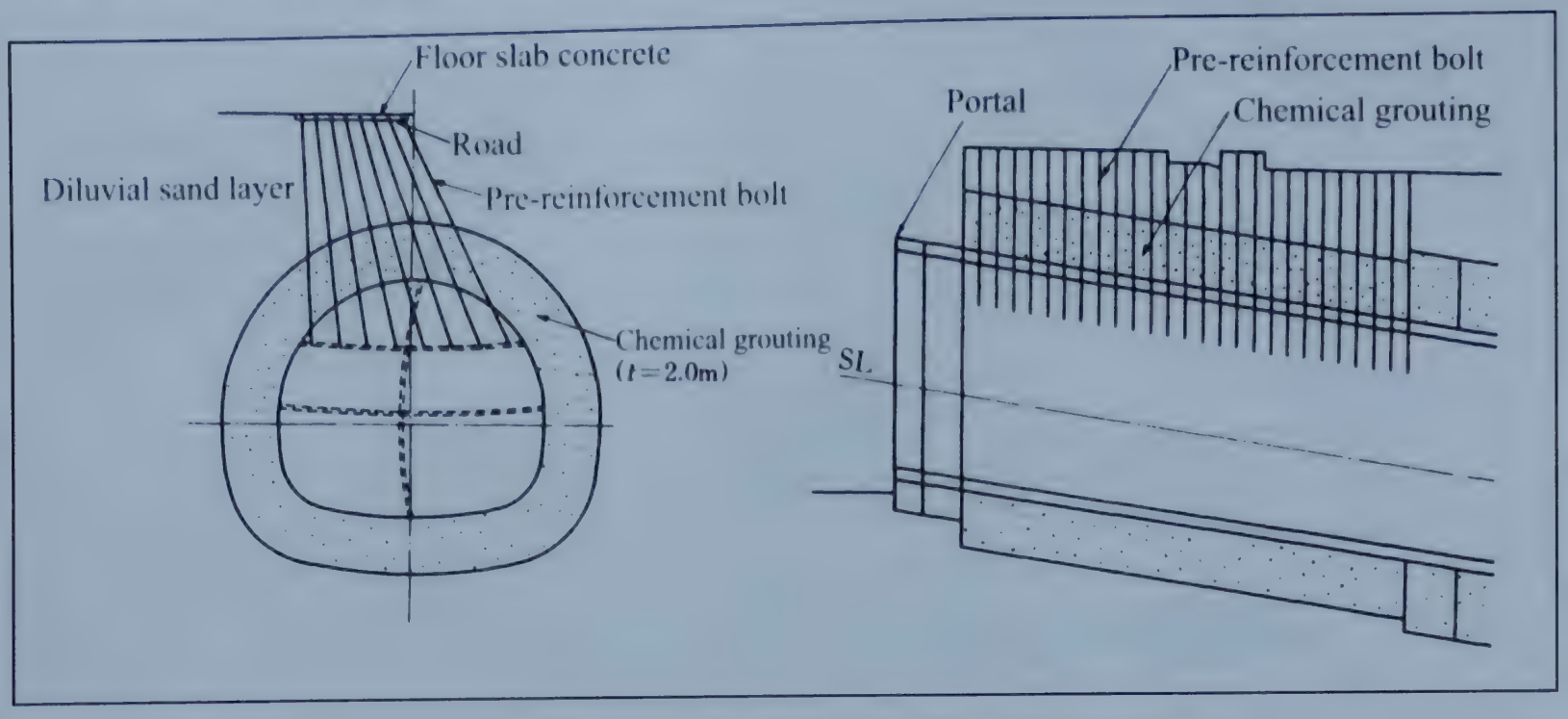


Figure 3-19: Example of Vertical Bolts and Chemical Grouting

3.3.2 Reinforcement Works for Existing Structures in the Vicinity

Reinforcement works used for protecting structures in the vicinity can be classified into methods to reinforce or improve the ground, cut-off wall methods, and methods to reinforce the existing structures. There are reinforcement/improvement works for the improvement of the ground such as chemical grouting or jet-grouting mixing which prevent the deformation of the ground. The ground surrounding the tunnel may be not only damaged depending on soil condition, the order of construction, and the distance, but structures in the vicinity may be damaged by the pressure of grout or leakage of grout materials. Therefore, care should be taken when using these methods. Additionally, steps should be taken to prevent the contamination of groundwater in the area.

The cut-off wall is a technique for preventing the propagation of displacement or lowering of groundwater table by isolating an existing structure from the tunnel. According to the type of cut-off wall, this method can be subdivided into steel pile sheeting, pipe sheeting, jet mixing grout, and steel pipe piling. If the groundwater flow is cut off by these methods, care needs to be taken to protect groundwater environments. With this solution, there is a fear that the cut-off wall would cause discontinuity of ground deformation. Therefore, great care has to be taken not to exert adverse effects on the stability of the face during tunnelling as well as not to increase the deformation of the tunnel during construction. The methods for protecting existing structures can be divided into direct reinforcement methods or underpinning methods. Figure 3-20 shows cut-off walls built on both sides of a tunnel passing under an existing bridge, designed to achieve water sealing in order to prevent the lowering groundwater table.

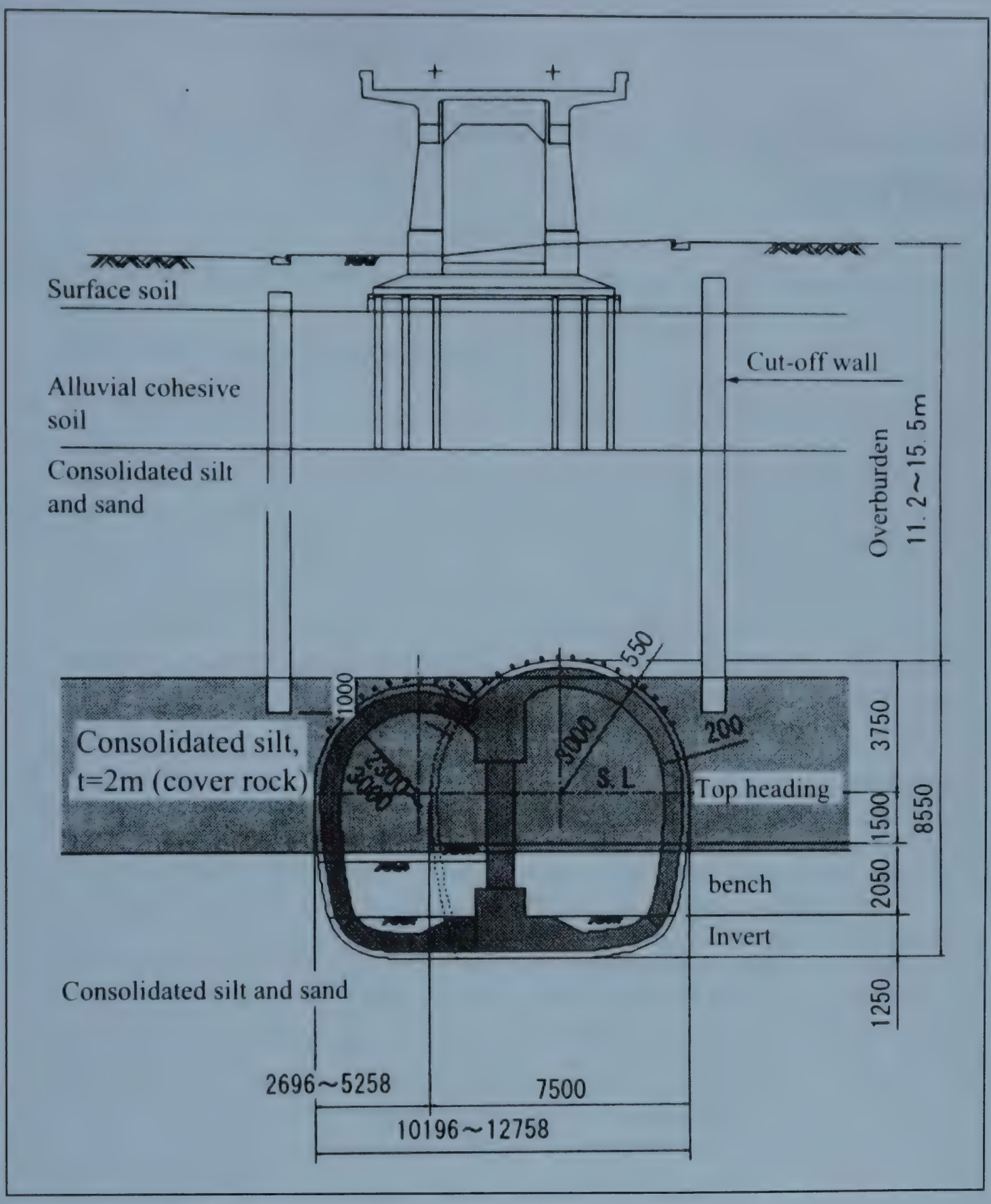


Figure 3-20: Example of Cut-off Walls for Water Sealing

CHAPTER 4 TUNNELING IN PECULIAR CONDITIONS

4.1 Difficult Ground Tunnelling

4.1.1 Tunnelling in Soil

(1) Groundwater Control

In aquiferous soil, where tunnel face often fails, ground stabilization methods for the face (including drainage methods and chemical grouting) need to be considered. In addition, consideration needs to be given to the impact on the surrounding environment (such as the ground surface settlement and water shortages caused by excavating the tunnel).

Therefore, when designing and constructing tunnels in the soil, surveys of topography, geology, groundwater, and the surrounding environment should be conducted in advance. After these surveys, methods for stabilizing the face, enhancing the bearing capacity of the ground, and preserving surrounding environments should be selected, based on predictions of ground behaviour around the tunnelling (either by means of analytical methods or referring to similar examples).

When constructing tunnels in soil containing groundwater, dewatering methods such as deep wells are often used in order to stabilize the face but extra care should be taken not to remove sediment with the water. In addition, refer to Methods for Face Stabilization for stabilization methods for the tunnel face or crown and refer to Ground Surface Settlement for information on ground settlement due to shallow overburden.

Water inflow may be caused by the rising of groundwater after cessation of dewatering or recovering groundwater accompanied by the completion of the lining. In this case, if fine soil particles are washed out, it may seriously destabilize the ground. In addition, a void may occur under the invert if fine particles of ground flow into the drainage pipe when the water catchment system uses a centre drainage pipe under the invert. Therefore, filtering materials should be used. For example, the centre drainage pipe should be placed either on the upper side of the invert, or the central pipe should be non-porous. Table 4-1 shows examples in soil and Figure 4-1 shows an example where the central pipe under the invert is non-porous.

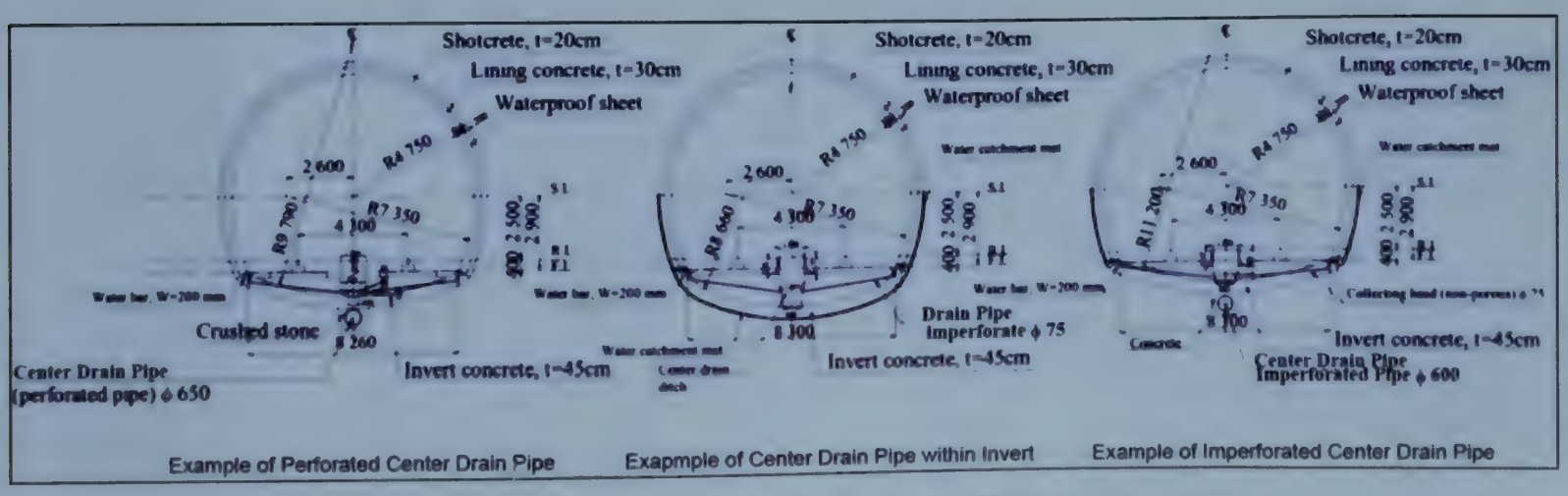


Figure 4-1: Examples of Centre Drain Pipes (Railway Tunnel)

(2) Excavation Method

Generally, when excavating the soil with little natural ground strength, the ground around the tunnel may create yielded plastified zone by loosening due to excavation (and the ground arch mechanism may not provide sufficient support where is shallow overburden). Therefore, methods for soil should be based on closing the perimeter of the cross-section, and loosening of the ground should be avoided as much as possible.

There are two main approaches to excavating soil. The first is partitioning the face into smaller parts in order to enhance face stabilization and prevent loosening the ground as much as possible. The second approach is to reinforce the perimeter of the cross-section ahead of the face by pre-supports (and/or other similar methods) in advance, in order to excavate as large an area as possible while preventing the loosened ground from dispersing outside the reinforced area by means of countermeasures.

(3) Tunnel Supports

Displacement of ground should be constrained at an early stage for soil. Therefore, supports, which are sufficiently rigid and are capable of rapidly producing excellent hardening effects by internal pressure, should be selected.

(4) Protection against High Water Pressure

In the case of aquiferous soil, excavation is often carried out while dewatering. However, groundwater can rise after the completion of the lining. In this case, appropriate drainage should be designed in order to protect the lining from the groundwater pressure and the cross-section should have a shape as smooth as possible in order to prevent concentration of stresses. Additionally, the perimeter of the cross-section should be closed with lining. If permanent drainage cannot be maintained (due to conditions in the locations etc.), the lining structure shall be sufficiently waterproof for local water pressure.

Table 4-1: Examples of Tunnels in Soil in Japan

Tunnel	Length *1 (m)	Geology	Over-burden (m)	Excavation Section	Excavation System	Supports			Auxiliary method	Lining Thickness (cm)	Notes
						Shot crete (cm)	Rock Bolt (nos)	Steel rib, Spacing			
A	2446	Diluvium Volcanic cohesive soil	1 - 16	Horseshoe 87 m²	Short bench	25	14	H-150 – H-200 1.0 m	Drainage boring, steel pipe force-piling, foot reinforcement pile, ground improvement at crown, ground grouting at foot of support.	30 - 40 (Steel fiber reinforcing)	Soil exists from the portal to about 800m
B	1443	Diluvium pyroclastic flow, silt	≥ 8	Horseshoe 87 m²	Short bench	25	10	H-200 1.0m	Deep well, forepoling, steel rib with wing rib, steel foot plate	30 (plain)	Soil exists from the portal to about 700 and 900 m.
C	4012	Diluvium sand / silt	≥ 6	Horseshoe 87 m²	Short bench	20	12	H-150 1.0 m	Grouting type forepoling	30 (plain)	Soil exists for 40 m long at about 1.480 km from the portal.
D	665	Diluvium	≤ 20	Horseshoe 76 m²	Short bench	10 -15 (TH) 20 (Bch)	4 (TH) 4 (Bch)	Top Heading: H-100, H-150 1.0-3.0m, Bench: H-150 1.0-1.5m	Slit concrete	30 (plain)	Construction interval is by slit concrete installation
E	2010	Diluvium, Pumice flow deposit	5 - 25	Horseshoe 81 m²	Short bench	15	8 (3.0m)	H-125 1.0 m	Drainage drift, vertical pre-reinforcement bolt.	30 (plain)	Drainage drift using TBM (φ3.28m)
F	5178	Pliocene sand	200	Horseshoe 81 m²	Short bench	15	12 (3.0m)	H-125 1.0 m	Drainage drift (detour branch), grouting type forepoling, face shotcrete, face bolt.	30 (plain)	
G	1072 (1035)	Pliocene volcanic sandy and cohesive soil	5 - 25	Horseshoe 81 m²	Short bench	15	12 (3.0m)	H-125, H-150 1.0 m	Drainage boring, forepoling.	30 (plain), 45 (reinforcing)	
H	3923 (3883)	Neogene, mudstone, sandstone, siltstone.	0 - 178	Horseshoe 80 m²	Short bench	12.5 - 20	6 -14 (3.0-4.0 m)	H-125, H-150, H-200 1.0-1.2 m	Steel pipe forepoling, foot reinforcement pile	30 (plain), 30-45 (reinforcing)	
I	1205 (988)	Diluvium, Ito volcanic deposit	2 - 50	Horseshoe 67 - 77 m²	Short bench	10-15	(Additional rock bolts)	H-100, H-125, H-150 1.0 m	Additional rock bolts, well point drainage	30 (plain), 30-40 (reinforcing)	
J	1150	Diluvium, Ito volcanic deposit	2 - 23	Horseshoe 73 - 77 m²	Short bench	12.5 - 15	(Additional rock bolts)	H-125, H-150 1.0 m	Forepoling, well point drainage	30 (plain), 30-40 (reinforcing)	
K	2498 (2380)	Diluvium, Ito volcanic deposit	0 - 55	Horseshoe 73 - 84 m²	Short bench	12.5 - 20	(Additional rock bolts)	H-125, H-150, H-200 1.0 m	Forepoling, well point drainage, Chemical grout.	30 (plain), 30-60 (reinforcing)	
L	340	Diluvium, roam, scoria, sand	≤ 17	Chestnut 143 m²	Short bench	25	14 (6.0 m)	H-200 1.0 m	Forepoling, horizontal jet grouting	45 (plain)	
M	813	Diluvium, clayey conglomeratic sandstone, conglomerate.	22.5 (average)	Horseshoe 133 m²	Short bench	25	10 (6.0 m)	H-200 0.75 m	Forepoling	45	Three lanes (Inbound lane)
				Horseshoe 94 m²	Short bench	25	8 (4.0 m)	H-150 1.0 m	Forepoling	35	(Two lanes (outbound lane)
N	1282	Upper cretaceous gravel (Colluviums; Decomposed granite soil)	50 (average)	Horseshoe 94 m²	Short bench	20	19 (4.0 m)	H-150 1.0 m	Grouting type forepoling, steel pipe forepiling, face shotcrete, face bolt.	30	Outbound lane.

4.1.2 Swelling Rocks

(1) Prediction of Swelling

In swelling rocks, it is known that the behaviour of rocks around the tunnel and the earth pressure show singular phenomena. For example, shotcrete linings are gradually squeezed on the inner side. In extreme cases, the cross-section shrinks so much that the tunnelling project cannot be completed. Characteristically, displacements by squeezing are found at the crown and side of the tunnel as well as the bed and tunnel face.

When tunnel support systems are used to prevent displacements, a large earth pressure acts on them. Even the earth pressure is small just after excavation, but it increases later (and may eventually destroy the support). Additionally, when creep strain accumulates over a number of years, the concrete lining or invert can be damaged and may result in the tunnel having to be reconstructed.

As mentioned above, the behaviour of swelling rocks is different from the ordinary ground. Therefore, the distribution and characteristics of the swelling rocks shall be determined (as far as possible) by investigation. In addition, appropriate designs, tunnelling methods, and visual inspection and monitoring should be carried out. In particular, appropriate drainage should be taken because swelling or slaking occur in particular rocks such as Neogene mudstone and tuff affected by groundwater and by the release of earth pressure. Table 4-2 shows examples of tunnels in swelling rocks.

Figure 4-2: Squeezed Tunnel Section by Rock Swelling[1]

(2) Tunnelling Method

In swelling rocks, conditions for face stability are likely to be poor. The ground around the tunnel can loosen greatly and a large amount of earth pressure acts on the tunnel. Measures should be taken to minimize the loosening of the ground around and ahead of the face, and it is important that the perimeter of the cross-section should be closed by supports as soon as possible after excavation.

The standard excavation method is usually the short bench cut method; such as the mini bench cut method or the full-face method with bench cut, in which the length of the bench is shorter than usual.

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[1] Satoshi Kitayama, Setsuo Ide (1982); NATM in weak tuff rocks, Oritsume Tunnel, Tohoku Expressway, Tunnel and Underground Vol.13 No.8

Table 4-2: Examples of Tunnel in Swelling Rocks in Japan

Tunnel	Length (m)	Shape (Width of inner section) (m)	Kind of rock	Excavation system (Interval)	Support			Lining thick-ness (cm)	Notes
					Shotcrete (cm)	Rock bolt (length nods)	Steel support (Spec. spacing)		
Seikan	53 850	Circle (9.80)	Mudstone, Tuff	Bench Cut, Pilot drift (partial use) (446 m)	35	6-11 m 17	ϕ 267.3-318.5 (0.7-0.9 m)	70-90	Rock bolt: self drilling type (partial use) Steel support: lattice girder (partial use)
Nakayama	14 830	Horseshoe	Mudstone, Tuff	Bench Cut (800 m)	15-20	3-6 m 17-35	MU-29 (0.9-1.0 m)	50	
Nabetachi-yama	9 117	Circle (7.20) Ovoid (4.92)	Mudstone	Bench Cut, Drift Advancing (partial use) (2 296 m)	15-37.5	3-5 m 6-40	H-150,H-175 MU-29 (0.5-1.0 m)	30-60	Grout for improvement of the ground (partial use)
Komadome	2 000	Horseshoe (8.50)	Green tuff	Bench Cut (1 200 m)	15-20	2-6 m 14-27	MU-29 (0.8-1.2 m)	50	
Enrei	5 994	Horseshoe (8.54)	Mudstone	Bench Cut (2 000 m)	10-30	2-8 m 22-44	H-150,H-200 MU-29(1.0 m)	30	
Shin-Usami	2 985	Circle (6.00)	Solfataric clay	Bench Cut (2 985 m)	10-20	2.5-5 m 16-24	MU-29 (0.8-1.4 m)	30	Shotcrete: Steel fiber reinforced (partial use)
Enasan II	8 635	Horseshoe (10.20)	Hydrothermal Metamorphic Granite	Bench Cut (350 m)	25	6-9 m 22-61	MU-29 (1.0 m)	30-45	Additional rock bolts (span of 13.5 m) Shotcrete: Steel fiber reinforced (partial use)
Orizume	In-bound lane 2 321 Out-bound lane 2 272	Horseshoe (10.20)	Muddy tuff	Bench Cut, Drift Advancing (partial use) (Inbound lane 950 m Outbound lane 1 033 m)	20	6-9 m 43-51	H-200 (1.0 m)	30-50	Shotcrete: Steel fiber reinforced (partial use)
Nakaya	1 260	Horseshoe (10.40)	Tuff	Bench Cut (210 m)	20-25	4-9 m 18-58	H-150,H-200 (0.6-1.0 m)	30-50	
Funagata	1 368	Horseshoe (11.40)	Mudstone	Bench Cut (270 m)	25	4-6 m 26	H-200 (1.0 m)	30-50	
Nikkure-yama I	2 223	Horseshoe (10.20)	Mudstone	Bench Cut, Drift Advancing (partial use) (380 m)	15-35	4-6 m 16-32	H-125,H-150, H-200,H-250 (1.0 m)	30-80	Shotcrete: Steel fiber reinforced (partial use) Lining: Steel fiber reinforced (Grout for improvement of the ground, partial use)
Iwate	25 810	Horseshoe (10.20)	Tuff Tuffaceous sandstone Tuff breccia	Bench Cut (940 m)	20-30	3-4 m 12 -44	H-125,H-150, H-200 (1.0 m)	30	Shotcrete: Steel fiber reinforced(partial use) Lining: Steel fiber reinforced and strut for invert (partial use)
Iiyama	22 225	Horseshoe (9.50)	Sandstone Mudstone	Bench Cut (1 931 m)*1	25 (the 1st support) 12.5-15 (the 2nd support)	4-6 m 14-36	H-125,H-150, H-200 (1.0 m)	30	Shotcrete: Steel fiber reinforced (partial use) Multiple-layer supports
Taroyama II	4 264	Horseshoe (10.20)	Tuff	Bench Cut (594 m)	15-30	4 m 18	H-125,H-150, H-200 (1.0 m)	30	Shotcrete: high strength shotcrete (partial use) Lining: Steel fiber reinforced (partial use)

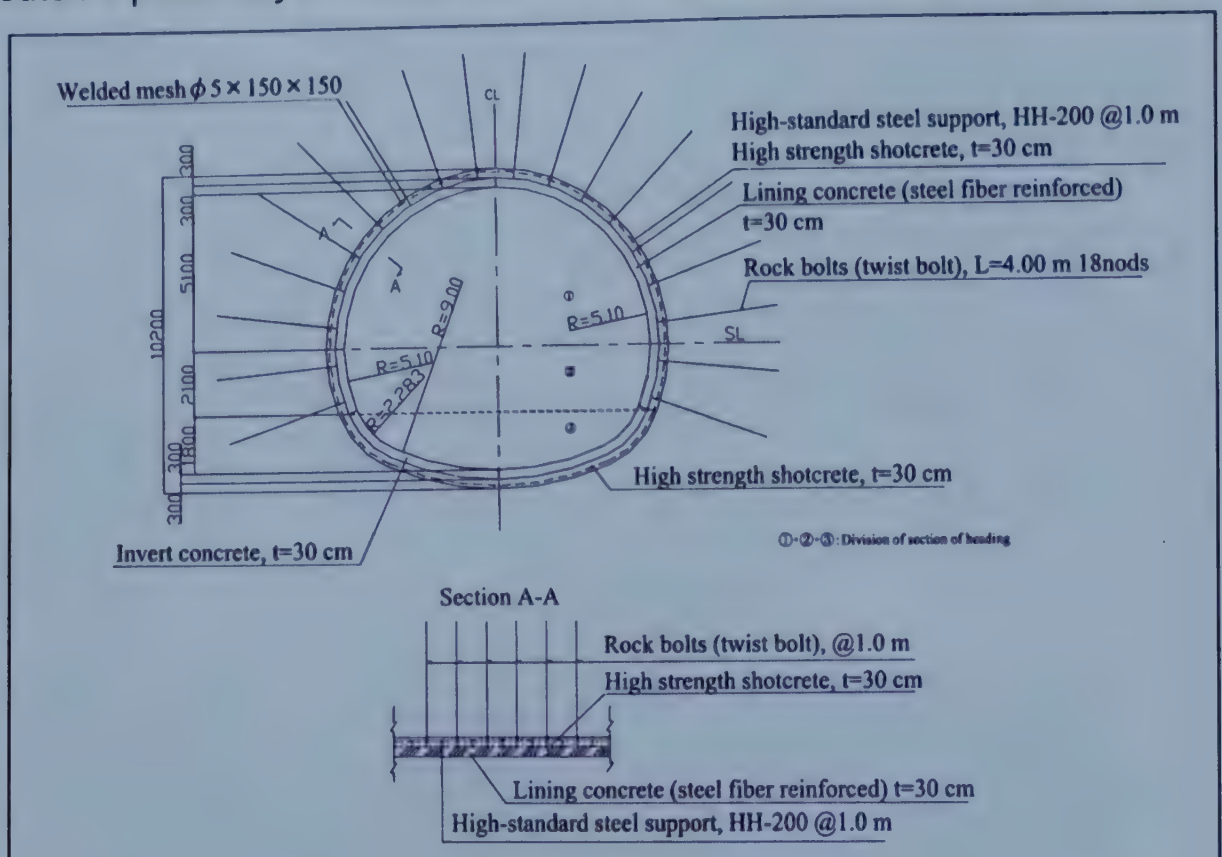
*1 describe the interval using multiple-layer supports

(3) Tunnel Support

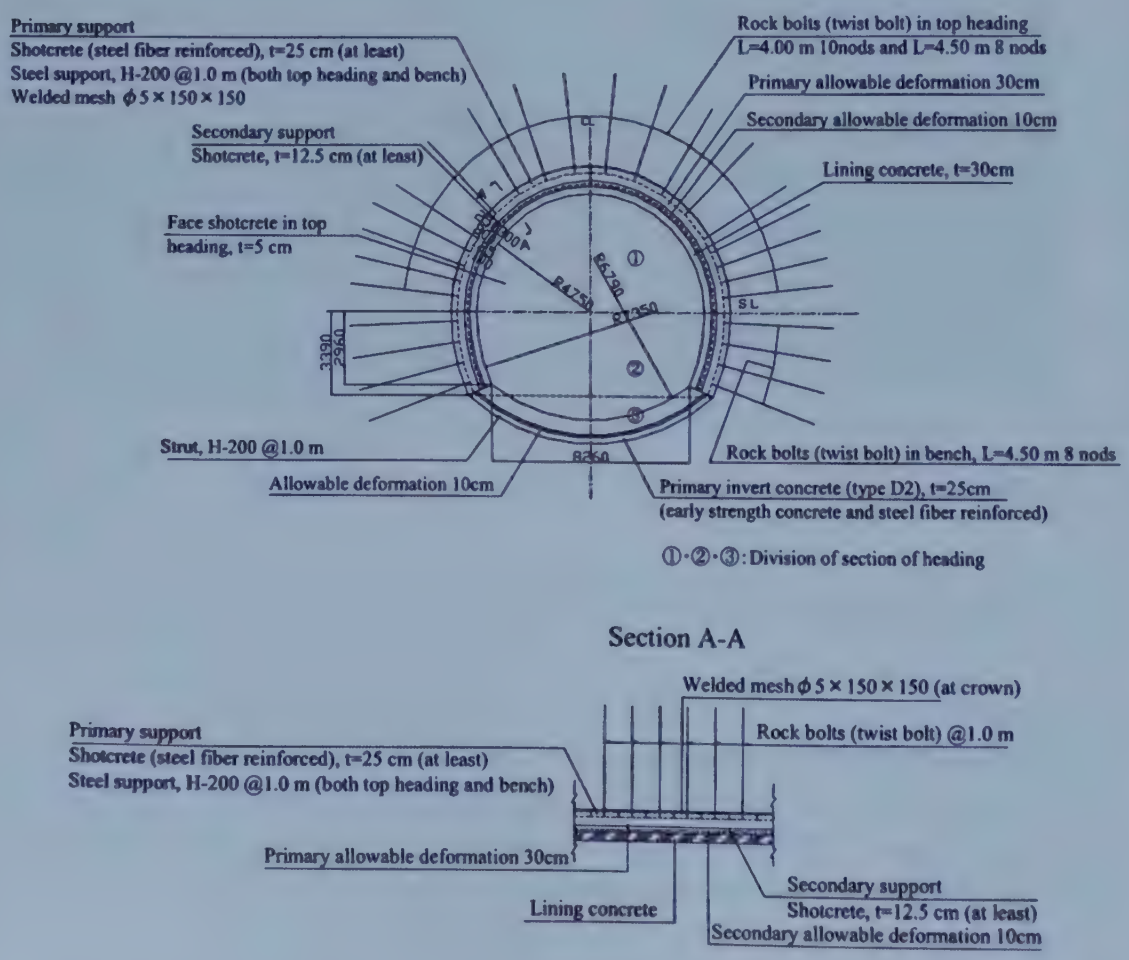
In swelling rocks, the countermeasure applied should ensure that supports would not need to be replaced whatever ground behaviour is encountered. For example, the method using supports, which have more stiffness than usual selected for reducing the displacement of the ground. (Figure 4-3(a)). Despite this (and whatever materials and profiles are used for supports), full bearing capacity and ability to constrain movement of ground should not be predicted before closing the perimeter of the cross-section should ideally be closed after by supports that combine shotcrete, rock bolts, and steel ribs even if the primary support fails (allowing some degrees of ground displacement) (Figure 4-3(b)).

(4) Tunnel Lining

In swelling rocks, the cross-section should be made as close to circular as possible and the whole perimeter of the cross-section should be closed by lining, which contains an invert. The invert is often closed by concrete. However, when it is necessary to close the perimeter of the cross-section at an early stage, it is recommended that the temporary invert should be placed by shotcrete, which is easy to work and develops strength rapidly.



(a) Example of a temporary invert and high-standard steel support


Primary support
Shotcrete (steel fiber reinforced), t=25 cm (at least)
Steel support, H-200 @1.0 m (both top heading and bench)
Welded mesh ϕ5×150×150
Secondary support
Shotcrete, t=12.5 cm (at least)
Face shotcrete in top
heading, t=5 cm
Rock bolts (twist bolt) in top heading
L=4.00 m 10nods and L=4.50 m 8 nods
Primary allowable deformation 30cm
Secondary allowable deformation 10cm
Lining concrete, t=30cm
R4750
R6390
R7350
SL
Strut, H-200 @1.0 m
Allowable deformation 10cm
Rock bolts (twist bolt) in bench, L=4.50 m 8 nods
Primary invert concrete (type D2), t=25cm
(early strength concrete and steel fiber reinforced)
①·②·③: Division of section of heading
Section A-A
Welded mesh ϕ5×150×150 (at crown)
Primary support
Shotcrete (steel fiber reinforced), t=25 cm (at least)
Steel support, H-200 @1.0 m (both top heading and bench)
Rock bolts (twist bolt) @1.0 m
Primary allowable deformation 30cm
Lining concrete
Secondary support
Shotcrete, t=12.5 cm (at least)
Secondary allowable deformation 10cm

(b) Example of a temporary invert and multiple-layer support

Figure 4-3: Example of Reinforcement Works in Swelling Rocks

4.1.3 Rock Burst

When excavating ground where a danger of rock bursts exists, the likelihood of a rock burst shall be assessed in detail ahead of time and necessary countermeasures are taken before excavating.

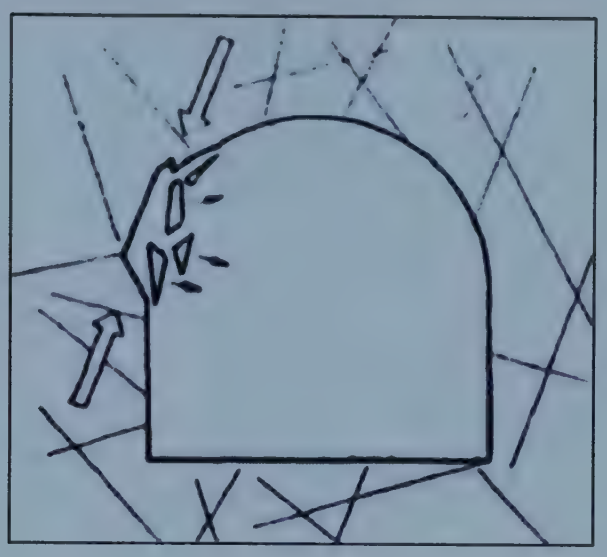


Figure 4-4: Mechanism of Rock Burst[2]

The prevention/protection measures against rockburst include:

- Covering the tunnel face with shotcrete or nets.
- Using friction bonded rock bolts in order to enhance the stitch effect or the suspension effect immediately after they have been driven into the ground.
- Using steel fibre-reinforced shotcrete, which can further improve support toughness, in order to reduce the danger of falling due to rockburst.
- Devising an evacuation plan for rock bursts.

When excavating ground where there is a danger of rock bursts, prediction of potential rock bursts is important. However, the detailed mechanism of rock bursts, prediction of potential rock bursts is important. However, the detailed mechanism of rock bursts is not yet known and precise predictive methods do not yet exist. In the meantime measurement by AE (i.e. acoustic emission) is sometimes used for studying rockburst mechanisms and predictions. Table 4-3 shows examples of tunnelling through the ground with the danger of rock bursts.

[2] Saqib Ahmad Saki (2013); Lahore Rock burst presentation, Lecturer at University of Engineering and Technology

Table 4-3: Examples of Rock Burst Occurred in Japan

Tunnel		Length (m)	Rock	Excavation section	Excavation system	Supports	Concrete lining	Location of rock burst	Countermeasure against rock burst
A		9702	Granite quartz diorite granodiorite hornfels	Single track 30 m²	Bottom drift top heading inverted lining method (Conventional tunnel method)	Timber lagging	30 cm (Precast)	Overburden: ≥ 1,000m Side wall	Stand by
B		13490	Granite quartz diorite granodiorite hornfels	Single track 35.4 m²	Full face (Conventional tunnel method)	Steel ribs (H-150), Rock bolts	30 cm	Overburden: ≥ 500m, Side wall	1) Protection for crown and side wall by 10 K rail, steel ribs (H-100, H-125) and timber lagging 2) Stand by
C		22221	Granite granodiorite	Double tracks (Sinkansen) 85.4 m²	Full face (Conventional tunnel method)	Steel ribs (H-200), Rock bolts	50 cm	Overburden: ≥ 500 m, Crown to side wall	1) Rock bolts 2) Rock fall prevention nets 3) Steel fibre reinforced shotcrete 4) Stand by
D Twin -tube	D1	10926	Quartz diorite hornfels	84.2 m²	Full face (Conventional tunnel method)	Rock bolts wire mesh	40 cm	Overburden: ≥ 750 m, Tunnel face	1) Shortening of round length (11.5 to 3 m is adjusted 1.2 m) 2) Face bolts and guard nets. 3) Change rock bolts to steel ribs and timber lagging 4) Stand by (Try to measure AE)
				86 m²		Steel ribs (H-200)	45 cm		
	D2	11020	Quartz diorite hornfels	85 m² (Two lanes)	Full Face	Rock bolts (L = 3m) Shotcrete (t = 5 cm)	30 cm	Overburden: ≥ 300 m, Crown and side wall	
E		6645	Granodiorite hornfels	60 m² (Two lanes)	Full face with auxiliary bench out	Rock bolt (L= 3 m) Shotcrete Steel ribs	30 cm	Overburden: ≥ 300 m, Crown and side wall	1) Steel fibre reinforced shotcrete (t=10 to 15 cm) 2) Friction bonded rock bolts 3) Steel ribs (H-150) 4) Stand by (Try to measure AE)
F		3900	Granite	71 m² to 164.8 m²	Full face	Rock bolts (L = 3m) Shotcrete (t = 10 cm)	30 cm	Overburden: 40 -100 m, Crown	1) Friction bonded rock bolts 2) Steel fibre reinforced shotcrete (t= 10 cm)

4.1.4 Geothermal Heat, Hot Springs, and Toxic Gases

(1) Prediction and Prevention

When designing tunnels through the ground with levels of geothermal heat, hot springs, and acid water, it is necessary to include countermeasures for acid corrosion of rock bolts, deterioration of concrete structures, and reduced durability of linings. In such conditions, there is a danger of the elevated curing temperature starting cracks in the concrete. In addition, it is thought that bicarbonate ions (HCO_3^-) in groundwater can corrode concrete. With regard to linings in these types of ground, the kind of cement, concrete mix, thickness, waterproof sheeting material, and curing method should all be selected to give maximum lining durability.

During the drill and blast construction of a tunnel in the ground with high levels of geothermal heat, there is the danger of charging explosives detonating spontaneously. Therefore, heat-resisting detonators and explosives, and/or mechanical excavation methods should be used. In the ground with hot springs, the issues mentioned above relating to high ground temperature should again be considered. There are the additional dangers of toxic gas eruptions, hot waters, and high-temperature vapour. In addition, the possible effects of the excavation on surrounding hot springs should be checked. To combat high temperatures, some tunnels use an air supply ventilation system combined with large ventilation facilities and cooling equipment.

(2) Countermeasures

Countermeasures for toxic gas and combustible gas should be prepared as follows:

- installation of portable and fixed gas detectors,
- installation of high-capacity ventilating facilities,
- use of explosion-proof electrical apparatus,
- fire control measures,
- routine monitoring of gas and oxygen levels etc.

In addition, members shall be checked, members to watch arranged, signs restricting access to authorized personnel posted worked trained, evacuation tools provided, and alarm systems and rescue systems shall be installed. Figure 4-5 shows an example of a management system for gas detectors in excavating ground where toxic gas is present.

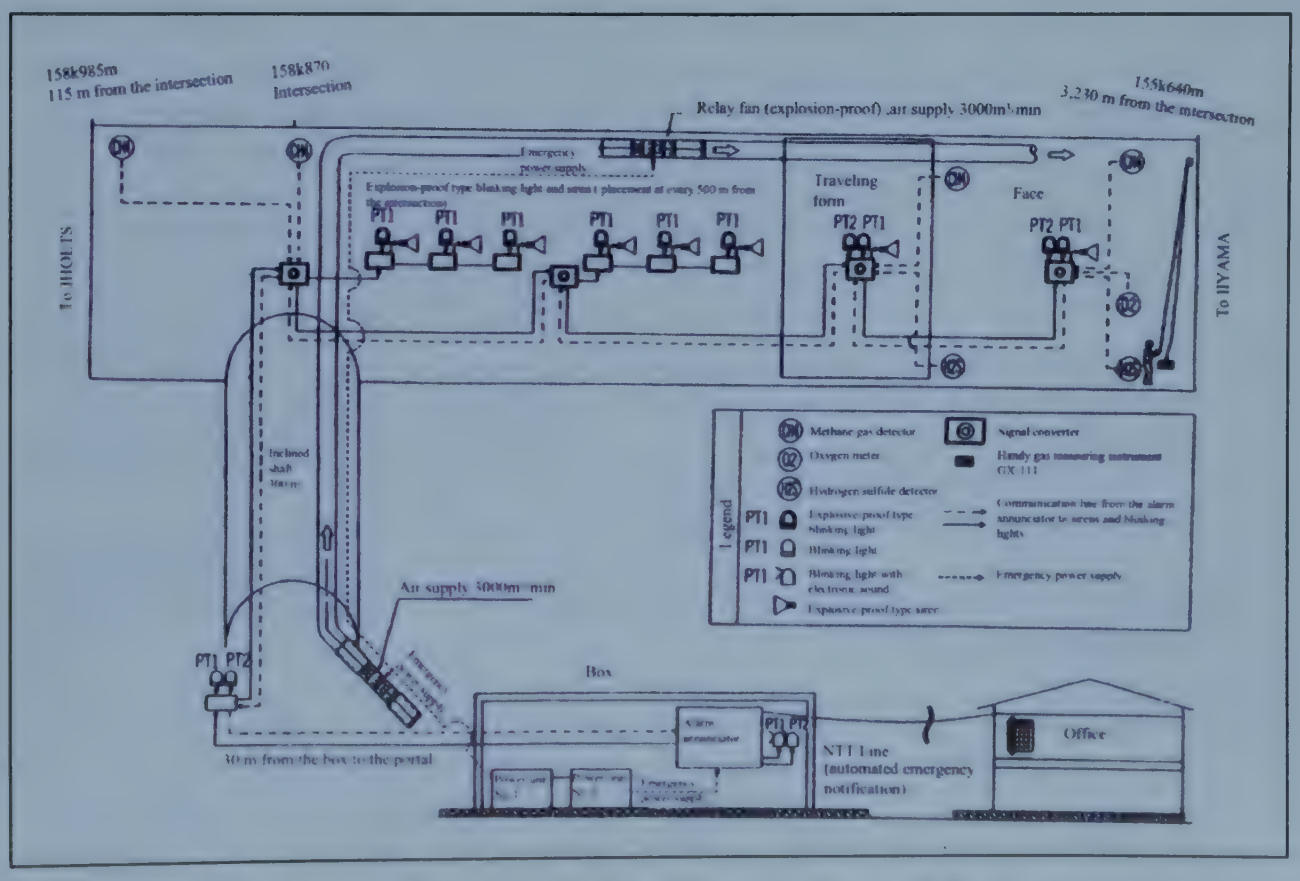


Figure 4-5: Example of Gas Detector Control System

4.1.5 High Water Pressure

(1) Prediction

In designing and constructing tunnels through the ground where there is a danger of high water pressure of large inflow volumes, a detailed survey should be conducted in advance of (and during) excavating. Measures should be taken to protect against unpredicted heavy water inflows.

Figure 4-6: Large Inflow during Tunnel Construction

(2) Prevention

If high water pressure of large inflow volumes is predicted, countermeasures appropriate for the ground conditions, aquifer, and circumstances of groundwater should be taken.

- Reducing water flow through draining or dewatering methods.
- Reducing the permeability or sealing by means of reinforcement by injecting grouting materials into the ground.
- Combination of two methods above.

4.2 Design of Portals on Peculiar Slopes

4.2.1 Problems in Portal Zone

Portal zones are the areas near the tunnel entrance and exit, where the overburden generally is small and a ground arch is not formed easily.

Portals are frequently located in a generally weak geologic zone, where erosion develops and a complicated topography is created. Thus, tunnelling or construction of the portal is likely to cause landslide or slope failures. While the tunnel is in service, portals are susceptible to natural and meteorological disasters such as rockfalls, avalanches, debris flows, and earthquakes. Portals sometimes may be subjected to overburdened loads of earth pressure after being placed into service.

Therefore, during the design of portal zones and portals, the design engineer must decide on an appropriate portal structure construction method, portal forms, portal shapes, and the portal structures by giving sufficient consideration to natural conditions such as the topographic features, geology, groundwater and weather in the vicinity of the portals and socially restrictive conditions, including the presence or absence of homes and structures. The engineer also must take into account factors such as slope stability, the possibility of natural disasters, harmony with the surrounding environment, and the effects of operation of rolling stock.

Table 4-4 lists problems predicted mainly during the construction of a tunnel portal zone and recommendations.

The portal zone generally is defined as an area where the depth of overburden is one to two times the excavation diameter where development of a ground arch is difficult, as shown in Figure 4-7. The scope of the zone, however, must be determined in each case by taking into consideration the ground conditions of the individual tunnel.

Table 4-4: Problems at Tunnel Portal Zones and Recommendation

Problem	Recommendation
Landslide	Working in the portal zone sometimes induces a slope failure or landslide. This can be attributed to loosening because of tunnel excavation or slope cut for the construction of the portal. If a slope failure or landslide by tunnel excavation is predicted, slope stabilization measures should be taken in advance of the tunnel portal construction.
Unsymmetrical earth pressure	Unsymmetrical earth pressure may act on the tunnel section and large stress may act on the tunnel depending on the position and relationship of the slope and tunnel. Measures are required to balance the earth pressure by embankment as a counterweight or cut for slope stabilization.
Soil and weathered rocks	The whole weight of the overburdened earth may load on the tunnel in the shallow depth tunnel. Soil or weathered rocks at the portal area may induce settlement or deformation of the ground surface. Measures to prevent ground settlement and defamation are required in soil and weather rock areas around the portal
Face collapse	the tunnel face near the portal could be unstable since the ground at the portal may consist of loose and soft soil and highly weathered rocks with many cracks. Excavation method and reinforcement works to stabilize the tunnel face shall be redesigned.
Ground surface settlement	In case that there is buildings or important structures on the surface above the tunnel, measures and reinforcement works to prevent any deformation on the ground surface shall be employed
Rock falls, debris flow, or avalanches	The portal zone must be designed for a location free from rockfalls, debris flows, or avalanches. If such positioning of the portal is impossible, adequate measures should be taken against disasters.
Structures in the vicinity	The Impact of tunnelling construction on existing structures in the vicinity such as houses, steel towers, roads, and railways, and the effect of noise and exhaust gas after the tunnel is placed into services, also should be studied.

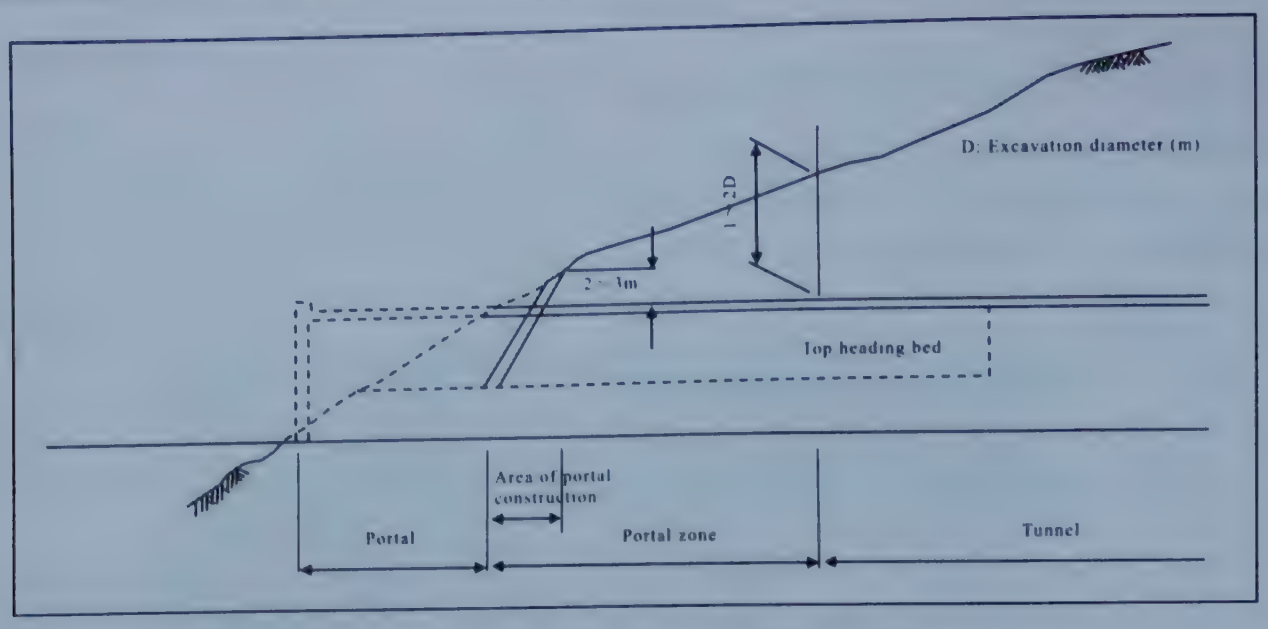


Figure 4-7: Area of a Standard Portal Zone

4.2.2 Tunnel Design in Portal Zone

(1) Design Concept

A tunnel portal zone is divided into a section for portal construction, where the slope is cut prior to the start of tunnel excavation, and a transition section for the tunnel (Figure 4-7). The portal is constructed by cutting while using shotcrete, rock bolts, or other measures to stabilize the natural slope, to ensure safe and simple tunnel excavation.

During the construction of a portal, the stability of the portal slope, the bearing strength of the ground, the relationship between the tunnel axis and the slope, the methods for portal excavation and tunnelling, and the compatibility with the surrounding landscape should be considered. From the standpoint of preserving the natural environment, a plan should be adopted that does not require a larger permanent slope behind the portal than necessary, and when a permanent slope may be left, natural vegetation should be used to create a green zone. When the portal slope is unstable, positioning the portal slightly forward in combination with counter-weight fill often produces a favourable condition. A deep cut of a slope to shorten the tunnel length is likely to cause slope failure or landslide, and therefore should be avoided.

(2) Tunnel Support in Portal Zone

Because of extensive talus, soil, and shallow overburden in portal zones, a ground arch is difficult to form. As a result, the entire load of the overburden might act on the tunnel as mentioned above, and the design of highly durable supports capable of resisting the acting load is required (refer to Design Concept for Tunnel Supports).

The concrete lining must be reinforced with the installation of an invert and reinforcing bars.

When the geology is extremely weak, the load must be estimated and the specifications must be decided based on the structural calculations.

Because deterioration of the lining material as a result of icicles or freezing and thawing caused by the effects of the outside air also may be problems in the portal zone, appropriated waterproofing, drainage and insulation should be applied as necessary for managing maintenance. Since an earthquake can easily have an adverse effect near the portal, a study of such effects should be conducted when ground conditions are poor.

(3) Problems in Portal Zone

Problems predicted during tunnelling works, such as landslides, insufficient bearing capacity of the ground, unsymmetrical pressure, ground surface settlement, and natural damage, should be comprehensively studied comparatively from the viewpoint of safety, ease of construction, workability, and economic efficiency. Appropriate corrective measures should then be designed.

Table 4-5 lists general solutions to the problems at the tunnel portal.

Table 4-5: Problems in Tunnel Portals and Solutions

Solution	Slope Failure	Landslide	Unsymmetrical pressure	Insufficient bearing capacity of ground	Face collapse	Ground surface settlement	Remarks
Slope protection	+	+					
Retaining wall	+	+	+	+			Enlarged retaining wall
Slope cutting or earth buttress on slope	+	+	+				Soil cement
Vertical reinforcement bolt	+	+	+		+	+	
Anchoring	+	+					Landslide prevention piles, ground anchors.
Drainage	+	+			+		Well point drainage, drainage boring, deep wells.
Ground grouting	+	+		+	+	+	
Forepoling					+	+	Pipe-roof, forepoling, long forepiling, horizontal jet grouting, slit concrete
Tunnel face reinforcement					+	+	Face shotcrete, face bolts, long face bolts.
Tunnel wall foot reinforcement				+	+	+	Steel ribs with wing ribs, foot reinforcement bolts, foot reinforcement piles, temporary invert

+: Typically used

4.2.3 Design of Tunnel Portal

(1) Design Concept of Tunnel Portal

The portal zone from the landslide, rock falls and avalanches down the slope surface, and requires a mechanically stable design. The external appearance and shape of the portal should be determined according to the use of the tunnel and in order to maintain compatibility with the surrounding natural environment and landscape.

(2) Items affect Portal Design

When determining the location and the type of portal, the following conditions shall be studied comprehensively.

- Topographic and geologic conditions (bearing capacity of the lower portion of the portal, stability of the surface behind the slope, presence of unsymmetrical earth pressure, the possibility of landslides of rock falls, positional relative to swamps and valleys)
- Meteorological conditions (snow, snowfall, snow cornices, drifting snow, avalanches)
- Positioning of the tunnel portal
- Environmental conditions (land use conditions, extent of changes in surrounding environment and landscape in the surrounding area)
- Coordination with plans for the vicinity (tie-in with open sections such as bridges, relationship with relocated roads, shifting channels, etc., location of planned future maintenance management facilities near the portal)
- Economic efficiency
- Other (direction of incident sunlight, etc.)

In particular, when selecting the portal location, care must be taken to not cut away the ground more than necessary, in order to ensure slope stability and preserve the natural environment.

Portal forms are classified as either wall type or protruding type. Characteristics of the general types of portals are shown in Table 4-6.

Table 4-6: Types and Characteristics of Tunnel Portals

	Wall type			Protruding type			
	Gravity / Semi-gravity	Wing	Arch wing	Half protruding, Parapet	Protruding	Split bamboo, Bell mouth	Reverse split bamboo, Reverse bell mouth
Shape							
Overview	Gravity type retaining wall placed about 10m forward of the portal position. Recently replaced by wing type and now is seldom adopted.	Placed as retaining wall by cutting slopes. Economically effective since tunnel length can be shortened. Suitable for most railway tunnels.	In contrast to wing type, the shape mitigates the extension of the tunnel length and round form.	Arch section created as a protruding type and an earth retaining wall added to prevent fill collapse. A reasonable structure as a portal.	A form used mainly to improve the landscape using open tunnelling and construction the portal so the inside section united with the tunnel is protruding.	The form eliminates the concrete exposed section of a protruding type portal, creating a bell mouth type portal with a trumpet-like opening.	Portal is larger than split bamboo type or bell mouth type, The form increases the concrete exposure of the upper section.
Ground Conditions	- Comparatively steep slope - Where structures like retaining walls are required. - Where rock falls are predicted.	- Comparatively steep slope where cutting and portal are proposed. -Where an earth retaining wall or enlarged retaining wall is constructed on oblique slope.	- Where topographic features are comparatively moderate - Where cut on both sides is comparatively small.	- Where there are ridge-like topographic features - - Where few structures on either side are connected to the portal.	- Where the surrounding topographic features are comparatively moderate. - Where an oblique type slope is difficult to correct. - Where counterweight fill works are used as slope measures. -Where rock falls are predicted.	- Where the surrounding topographic features are moderate and the area surrounding the portal is open. - Where the slope is perpendicular. - Where site is include landscaping around the portal.	- Where slope around the portal cannot be cut - Where rock falls are frequent.
Meteorological conditions	- Few problems even when used in snowy areas.	- Few problems even when used in snowy areas.	- Few problems even when used in snowy areas.	- Few problems even when used in snowy areas.	- Few problems even when used in snowy areas.	- Snow drifting and snow cornices occur easily when used in snowy areas.	- prevent snow drifting and formation of snow cornices in snowy area.
Important points for design and constriction	- Piles or a large-scale replacement foundation may required in accordance with geological conditions.	- Integration of the portal into the tunnel is required	- Opening tunnelling may be required depending on the topography (especially in arch sections) - A certain quantity of fill is required for slope protection.	- Open tunnelling several meters in the main tunnel is required, and some earth retaining walls are required to prevent fill collapse.	- Tunnel length can be extended -Protective fill normally required	- Form work and arrangement of reinforcing bars required much effort and cost - Protective fill normally required	- Adequate study of bearing capacity of the foundation required because of relationship to the centre of gravity point. - Form work and arrangements of reinforcing bars require much effort and cost
Landscape	- Because the wall has a large area, brightness must be reduced (for example, by chipping the wall surface) - it also makes drivers feel oppressed because the portal looks massive,	- Same as the gravity/semi-gravity type.	- Exertion of weight and pressure can be mitigated by the arch section curve. - Must consider compatibility with the surrounding topographic features.	- Exertion of weight and pressure is comparatively small because of a small wall area. - Compatible with the topography surrounding the portal	- Same as half protruding type/parapet type.	- Bell mouth type with widened entrance is easy to enter - Portal can be harmonized with surroundings topographic features by landscaping.	- Has little impact on travelling vehicles - Highly compatible with the topography surrounding the portal

(3) Design of Tunnel Portal

The portals shall be designed to ensure the portals are safe after the completion against dead loads such as backfill soil load or snow load, compaction load during backfill, wheel load when a road traverses the road upper section of the portal of the effects of earthquakes. It is necessary to combine these loads properly according to the type of portals.

Among wall-type portals, wing-type portals are normally designed as portals rigidly attached to the main body of the tunnel. When designing a wall as a retaining wall to receive the earth pressure behind the portal, the resultant wing stress can be calculated by assuming the main body of the tunnel to be fixed end and incorporating the earth pressure at the rest or Coulomb's active earth pressure, taking into consideration the effect of the backfill soil. Other loads frequently taken into consideration are the compaction load during backfill and snowfall load. The thickness of the wall should be the minimum thickness that can be constructed using reinforcing bar arrangement against the acting load, which in cases generally is assumed to be a thickness of approximately 50-70cm. Because the backfill earth pressure increases when the wall shape is large, lightweight fill material sometimes is used for part of backfill soil to control the thickness. Another approach is to make the rear of the wall independent using reinforced backfill.

For the design of a protruding type portal, the section force and bearing capacity of the ground shall be calculated by taking into consideration factors such as the dead weight of the portal itself, the surcharge from the fill, horizontal load, wheel load, and snow load. When part of the lining is exposed, the effects from earth pressure and inertia force during an earthquake, temperature changes, and concrete drying shrinkage, for example, must be considered. Normally, the section force is calculated using deformation analysis that considers the elastic spring for a combination of largest loads.

4.3 Tunnelling Close to Other Structures

4.3.1 Effect of the Tunnel on the Vicinity

(1) Effect of Tunnel on Structures in the Vicinity

Structures in the vicinity are above and below ground structures such as roads, railways, water channels, and buildings in the vicinity of a tunnel and buried structures such as gas lines and service water and sewerage conduits. Tunnel effects on neighbouring structures include deformation of the surrounding ground or blasting vibrations resulting from the tunnel excavation, and lowering of the groundwater table. For a description of the various effects on the surroundings environment other than the effects that a tunnel has on neighbouring structures, refer to Impact on Surrounding Structures and Environment.
If an effect on neighbouring structures during or after tunnelling works are predicted, a detailed study of those effects and control measures is required during the design phase. An adequate study must be completed beforehand because the extent of the effect on neighbouring structures may vary depending on factors such as ground condition, separation or relative location, size, work method, and work sequence.

(2) Mutual Effects of Neighbouring Tunnels

Neighbouring tunnels are two or more tunnels that are constructed simultaneously or in phases and affect one another. The case of later construction of originally unplanned tunnels in the vicinity of a completed tunnel is excluded.
Neighbouring tunnels are classified as follows.

- Parallel tunnels (When two or more tunnels lying side-by-side or above and below one another are constructed simultaneously or in phases. Two tunnels are referred to as twin tunnels.)
- Crossing tunnels (When two or more tunnels have an over-and-under positional relationship and intersect at a certain angle simultaneously or in phases.)
- Twin-tube tunnel (When twin tunnels, especially those having two or more tunnels, are extremely close together, share the centre bracing struts and all the tunnels are constructed at simultaneous periods. Referred to as a no-drift twin-tube tunnel when a twin tunnel is constructed without headings.)

Neighbouring tunnels are subjected to mutual effects of stress re-layout resulting from the construction of multiple tunnels. Depending on geological conditions, parallel tunnels that are separated from each other by a distance of two (in the hard ground) to five (in soft ground) times the diameter of the excavation at the tunnel centre interval are found to suffer very little mutual impact. For crossing tunnels, the required upper and lower offset distance varies greatly according to the geological conditions, tunnel size, tunnel intersection angle and positional relationship, work method, and work sequence. When the offset is small, the occurrence of detrimental mutual effects must be studied.
Blasting in the following tunnel may induce some problems in the advanced tunnel such as cracks and collapses of tunnel lining.

(3) Effects on Tunnel from Construction Works in the Vicinity

During the design phase, the influence on the tunnel of the construction works proposed near the tunnel be studied in the design stage. Because the extent of effects from construction works in the vicinity will vary depending on factors such as geological condition, offset or relative position, type and size of the construction work, the construction method and construction sequences, a prior complete study of the effects is necessary.
Stress on the supporting structures and surrounding ground that maintain tunnel stability can be predicted to change as a result of the construction works in the vicinity. Additional loads are likely to be placed on the tunnel, and displacement or deformation can be

induced, during and after the construction works. In some cases, a tunnel may be changed and its function as a tunnel may deteriorate as a result of this additional load or displacement and deformation. Because of the blasting vibration from the construction works, the generation of cracks in the supports or lining of the tunnel or chipping of the concrete during or after the construction can be predicted, the control measures must be studied.

4.3.2 Existing Structures

(1) Prediction of Effects

Effects on existing surrounding structures include ground displacement from excavation and ground settlement as a result of lowering the groundwater. In these situations, the factors that can affect structures and predict the behaviour of the structures or ground must be studied. Moreover, blasting vibration may have an effect on houses and facilities in residential areas. In each case, it is important to identify the factors that have an effect and predict the extent of the impact appropriately by referring to standard allowable values.

i) Prediction of effects from ground displacement

Prediction of the displacement or deformation of the ground and ground surface is made based on experience and analysis.

A theoretical analysis, which is a prediction of the amount of displacement and deformation using the theoretical formula for a circular tunnel in the semi-infinite elastic ground, is performed to provide a standard before construction.

Numerical analyses, such as the finite element method, are generally methods to integrate the tunnel and ground and analyse them in two or three dimensions. The method that can be used when modelling ground and structures include elastic, elastic-plastic, viscoplastic, and elasto-viscoplastic, and appropriate modelling of the ground conditions is important. Careful consideration also must be given to the ground physical properties values used for the analysis.

ii) Prediction of effects from lowering groundwater

In groundwater in which deposits such as alluvial clay strata are distributed, the problem of ground surface settlement is frequently found in addition to the lowering groundwater. The effects from groundwater lowering might differ depending on the effect from the ground displacement and will occur over the long term, and in some cases, continuous settlement may progress behind the tunnel face as well. In this case, the settlement prediction should be made using consolidation theory because of the number of changes in pore water pressure.

iii) Prediction of effects from blasting vibration

The normal and most economical method for excavating hard rock and medium-hard rock is excavation by blasting. Blasting operations generate vibrations that in some cases will affect existing surrounding structures. In general, sufficient consideration must be given control measures at the planning stage because the propagation route of the vibrations is complex, depending on the ground conditions and conditions of underground structures, and the reduction of vibrations during propagation is difficult. Generally, methods employing an empirical formula based on blasting powder quantity and vibration speed are used as a formula for making blasting vibration predictions. It is desirable to verify the prediction formulas during construction by means such as test blasting.

(2) Standard Allowable Values

Because standard allowable values are the basis for determining the extent of effects from construction projects, it is important to decide the values after adequate consultation with the parties concerned with other projects in the vicinity.

i) Approach to standard allowable values

Standard allowable values must be prescribed for the amount of deformation of neighbouring structures, stress, and vibration, taking into consideration factors such as the types of existing structures, their structural characteristics, their foundation forms, allowable stress, and extent of deterioration. In particular, the standard allowable values for the amount of deformation of neighbouring structures and stress must be set after sufficient study to ensure both the structural functions and structural safety as facilities.

ii) Method for setting standard allowable values

The standard allowable values for neighbouring structures to ensure their functions as facilities and secure their structural safety must be set by asking the managers of the neighbouring structures to provide the values. When the standard allowable values are not provided directly by managers, the values must be set by using historical results and conducting individual structural studies. The relationship between displacement rate and damage for blasting vibration is shown in Table 4-7, and the limit of deformation values for each foundation form by structure type are shown in Table 4-8. When applying these values, however, it is necessary to complete a sufficient study and confer with the managers of the neighbouring structures.

Table 4-7: Relationship between Displacement rate and Damage from Blasting Vibration

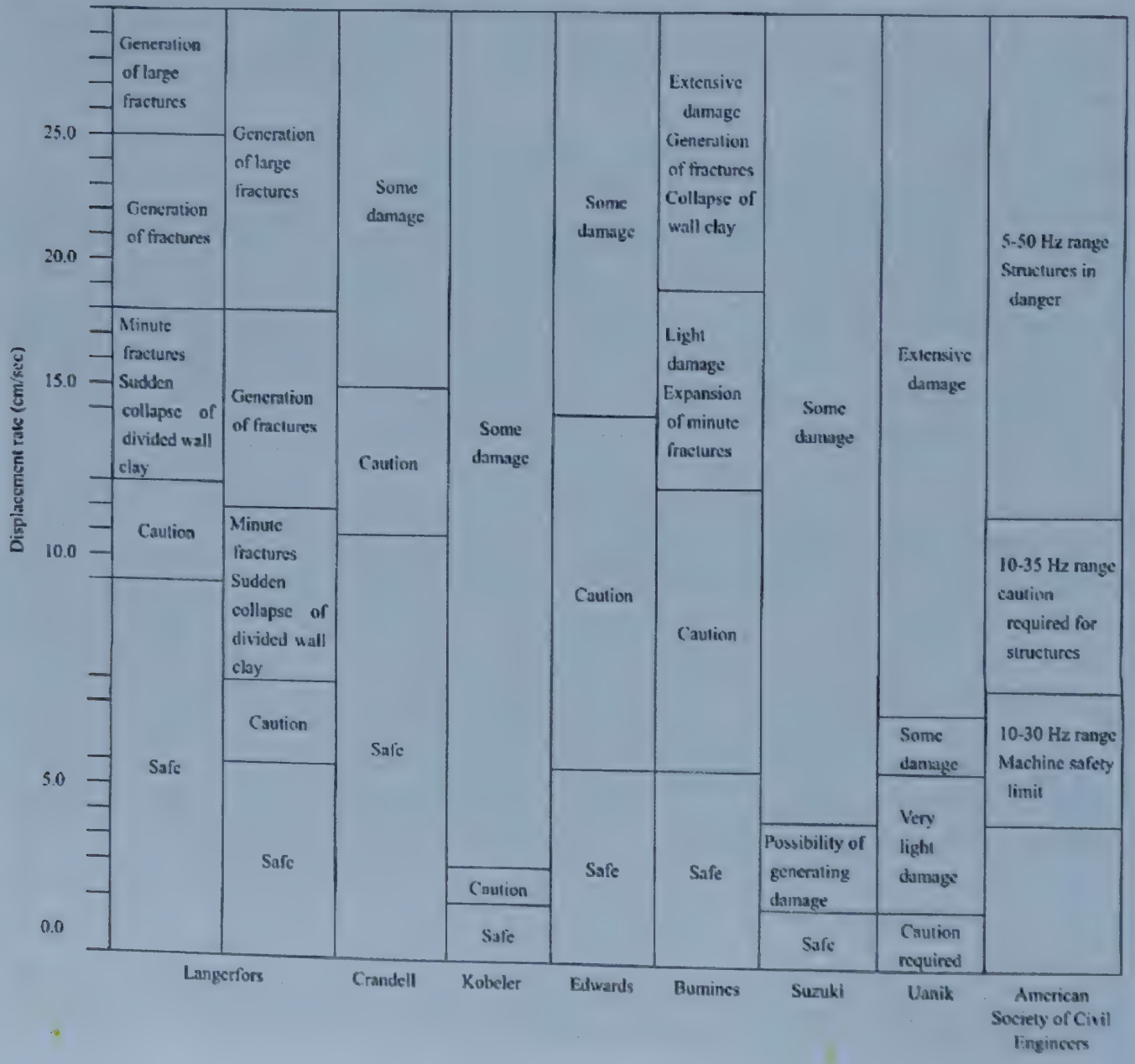


Table 4-8: Limit of Deformation Value for Each Footing Form by Structure Type

(a) Examples of limit of deformation angles by structure

Bearing ground	Structure type*1	Footing form	Lower limit of deformation angle*2 $\times 10^{-3}$ rad	Upper limit of deformation angle*3 $\times 10^{-3}$ rad
Consolidation stratum	RC	Independent, continuous, mat	0.7	1.5
	RCW	Continuous	0.8	1.8
	CB	Continuous	0.3	1.0
	W	Continuous	1.0	2.0-3.0
Weathered granite (decomposed weathered granite)	RC	Independent	0.6	1.4
	RCW	Continuous	0.7	1.7
Sand stratum	RC・RCW	Independent, continuous, mat	0.5	1.0
	CB	Continuous	0.3	1.0
Diluvial cohesive soil	RC	Independent	0.5	1.0
All ground	S	Independent, continuous (non-flexible finish)	2.0	3.5

*1 Abbreviations indicate the following structure types.
RC: Reinforced concrete construction, RCW: Wall type reinforced concrete construction, CB: Concrete block construction, W: Wood construction, S: Steel frame construction

*2 Lower limit of deformation angle: The deformation angle for which the number of sections where fractures are generated exceeds the number of sections where fractures are not generated, the deformation angle for which the probability of fracturing exceeds 50% or the deformation angle for which the total number of fractured sections exceeds 30%

*3 Upper limit of deformation angle: The deformation angle for which nearly all fractures will occur, where the total number of fractured sections exceeds 70%.

(b) Examples of relative settlement amount limit values by structure (Unit: cm)

Bearing ground	Structure type*	CB	RC・RCW		
Consolidation stratum	Footing form	Continuous footing	Independent footing	Continuous footing	Mat foundation
	Standard value	1.0	1.5	2.0	2.0-3.0
	Maximum value	2.0	3.0	4.0	4.0-6.0
Weathered granite (decomposed weathered granite)	Standard value	—	1.0	1.2	—
	Maximum value		2.0	2.4	
Sand stratum	Standard value	0.5	0.8	—	—
	Maximum value	1.0	1.5		
Diluvial cohesive soil	Standard value	—	0.7	—	—
	Maximum value		1.5		
All ground	Structure type	Finish material		Standard value	Maximum value
	S	Non-flexible finish		1.5	3.0
	W	Non-flexible finish		0.5	1.0

* Abbreviations indicate the following structure types.
RC: Reinforced concrete construction, RCW: Wall type reinforced concrete construction, CB: Concrete block construction, W: Wood construction, S: Steel frame construction

(c) Examples of total settlement limit value by structure (Unit: cm)

Bearing ground	Structure type*1	CB	RC・RCW		
Consolidation stratum*2	Footing form	Continuous	Independent	Continuous	Mat
	Standard value	2	5	10	10-(15)
	Maximum value	4	10	20	20-(30)
Weathered granite (decomposed weathered granite)	Standard value	—	1.5	2.5	—
	Maximum value		2.5	4.0	
Sand stratum	Standard value	1.0	2.0	—	—
	Maximum value	2.0	3.5		
Diluvial cohesive soil	Standard value	—	1.5-2.5	—	—
	Maximum value		2.0-4.0		
Consolidation stratum*2	Structure type *1	Footing form		Standard value	Maximum value
	W*3	Continuous Spread		2.5	5.0
				2.5-(5.0)	5.0-(10.0)
Immediate settlement	W*3	Continuous		1.5	2.5

*1 Abbreviations indicate the following structure types.
RC: Reinforced concrete construction, RCW: Wall type reinforced concrete construction, CB: Concrete block construction, W: Wood construction

*2 For consolidation stratum, value shows amount of settlement when consolidation is completed (calculation value disregarding building rigidity); for other ground, value shows amount of immediate settlement; values in parentheses ( ) show value when there is high sufficient rigidity such as double slabs

*3 Angle of inclination for W construction as a whole: standard 1/1,000, maximum of 2/1,000-(3/1,000) or less

(3) Control Measures

When the ground displacement, ground stress, and blasting vibration displacement rate for existing structures are predicted to exceed the standard allowable values, appropriate control measures corresponding to the scale of each value must be considered. It is important to select appropriate revisions to the excavation and excavation method or the control measures.

i) Ground displacement control measures

It is necessary to understand the cause of ground displacement and properly implement control measures corresponding to the cause. Depending on the ground properties, it will be possible to respond by revisions to the divisions of the tunnel face section or the excavation method. In the ground with a remarkable amount of deformation, however, in some cases, the natural ground strength can be reinforced by using face shotcrete and face bolts to control the extrusion displacement of the face or using oblique bolts and vertical pre-reinforcement bolts to reduce deformation of excavation crown. Refer to Reinforcement Works for Preservation of Surrounding Environment for details on methods used as a control measure.

ii) Groundwater lowering control measures

Water sealing methods such as chemical grouting, barrier walls, and freezing methods can be implemented as groundwater lowering control measures, based on the results of a study of ground condition and surrounding environment.

iii) Blasting vibration control measures

Blasting vibration reduction control measures include selecting an appropriate type of blasting powder, increasing the number of free faces, increasing the drill hole diameter (about 2 the decoupling coefficient), using non-electric percussion caps, use of pre-splitting, reducing the maximum amount of explosive and adopting mechanical excavation.

4.3.3 Proximity to Each Other

Neighbouring tunnels include parallel tunnels, crossing tunnels, and twin-tube tunnels.

(1) Parallel Tunnels and Crossing Tunnels

i) Mutual effects

For parallel tunnels and crossing tunnels, the behaviour of neighbouring tunnels varies substantially according to factors such as the distance between the tunnels, the relative position of the tunnels, overburden, ground condition, tunnel structure, construction method, and construction sequence. A prior study of the following items to identify the mutual effects on the tunnels is therefore necessary.

- Tunnel deformation
- Effects of groundwater
- Effects of blasting vibration

ii) Prediction of effects

Degrees of deformation and effects on supports vary according to factors such as ground condition, sections, and locations of neighbouring tunnels, distances between tunnels construction method, the structure of the advance tunnel, soundness, and functions required of the tunnels. It is important to identify the factors affecting both the advanced tunnel and second tunnels and evaluate the level of impact based on the classification of the proximity of the tunnel, and predict the deformation of the structures and the ground. Figure 4-8 shows an example of proximity classification.

When predicting effects, investigations and studies of similar construction cases should be supplemented by numerical analysis, theoretical analysis, and other methods whenever required.

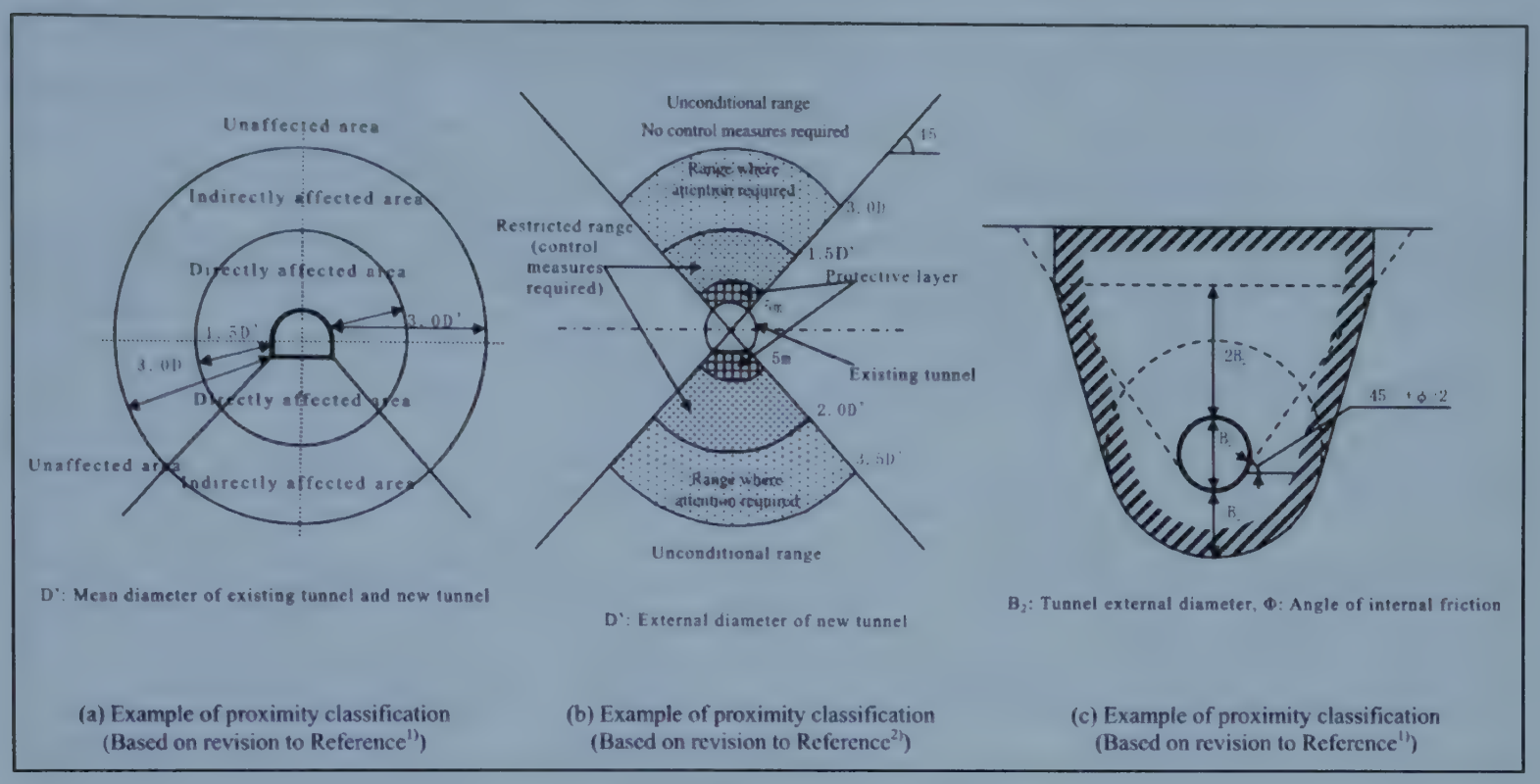
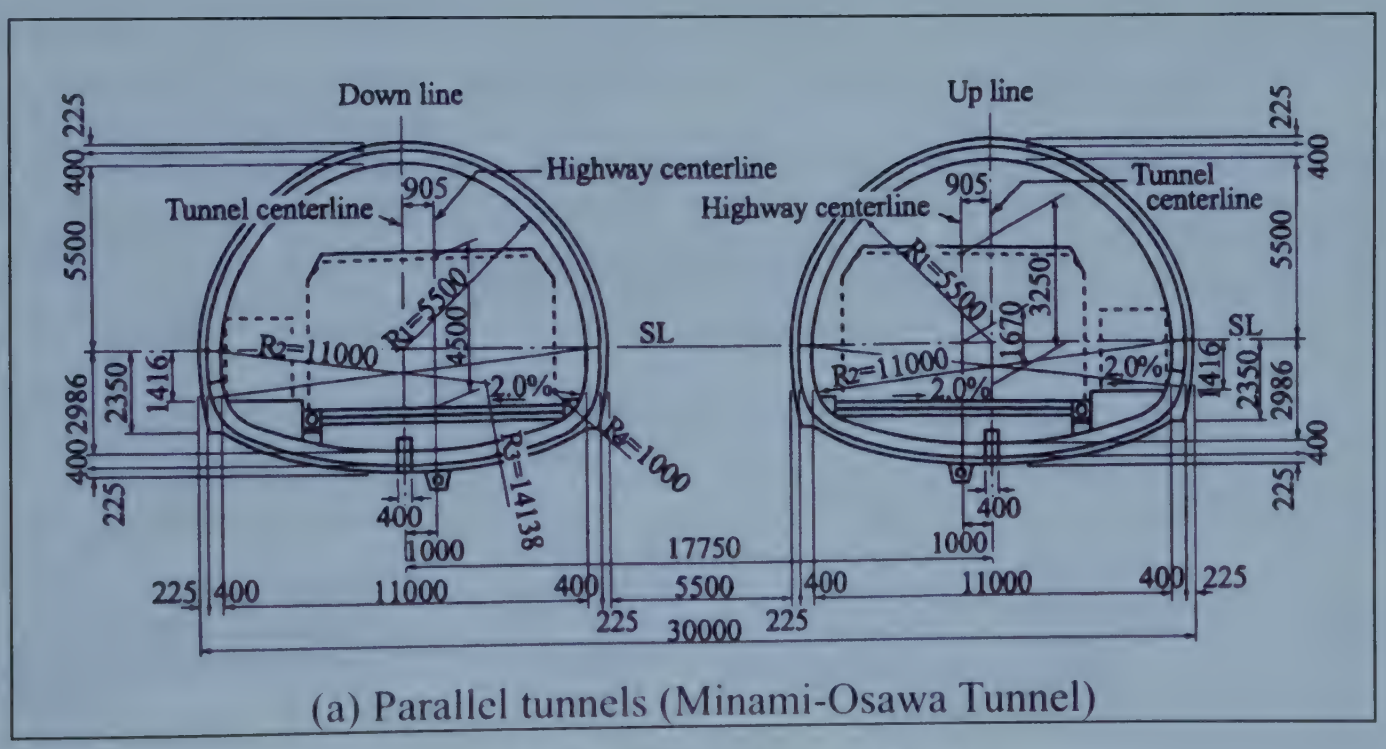


Figure 4-8: Example of Proximity Classification

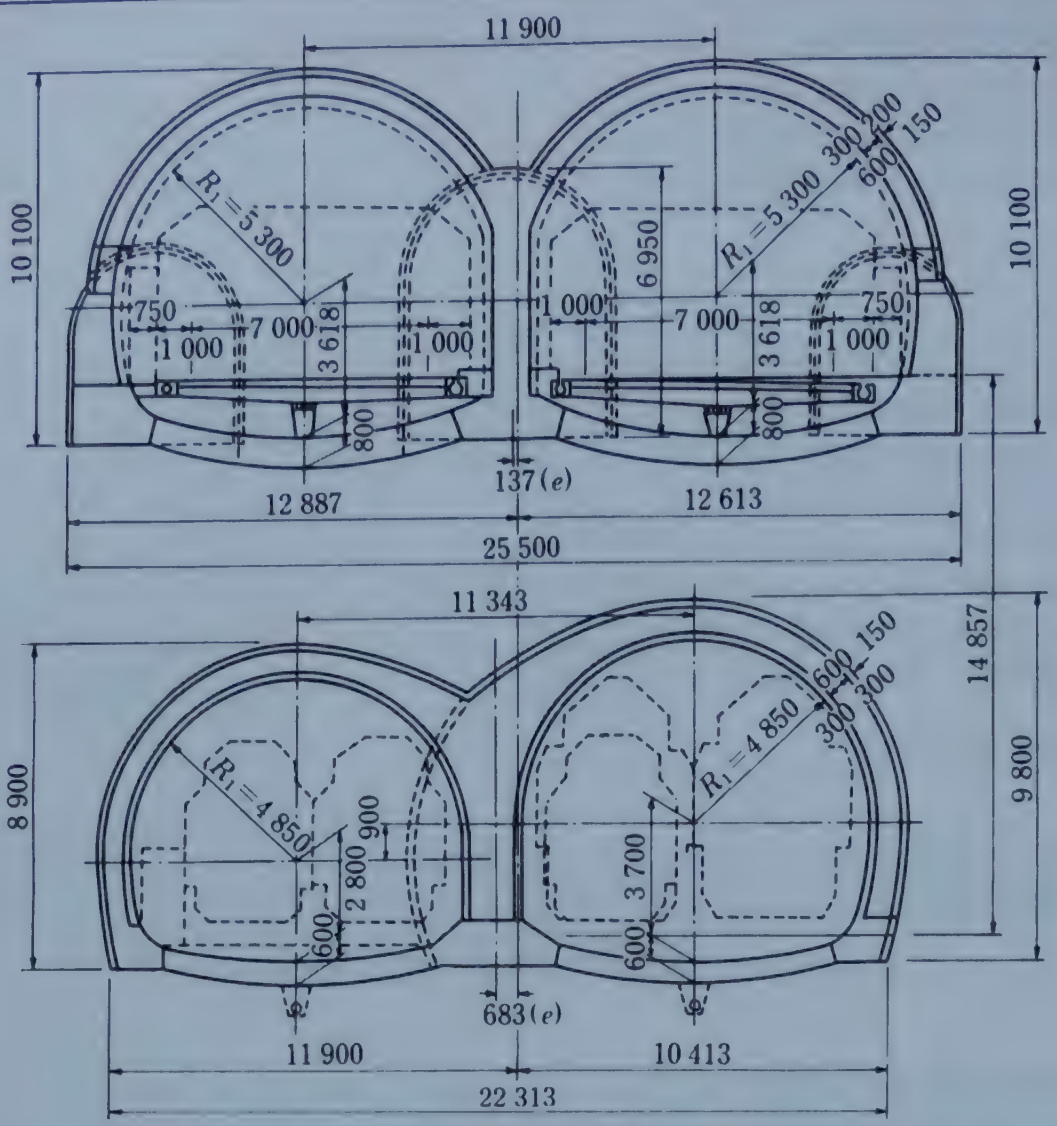
iii) Design of supports and linings

Construction of parallel tunnels and crossing tunnels requires the design of appropriate supports and linings based on the results of predictions of mutual effects. Especially when the advanced tunnel is predicted to suffer considerable deformation, measures such as strengthening the supports of the advanced tunnel built or reinforcement of linings with reinforcing bars are effective. Examples of the section shape of parallel tunnels and crossing tunnels are shown in Figure 4-9 and Figure 4-10.

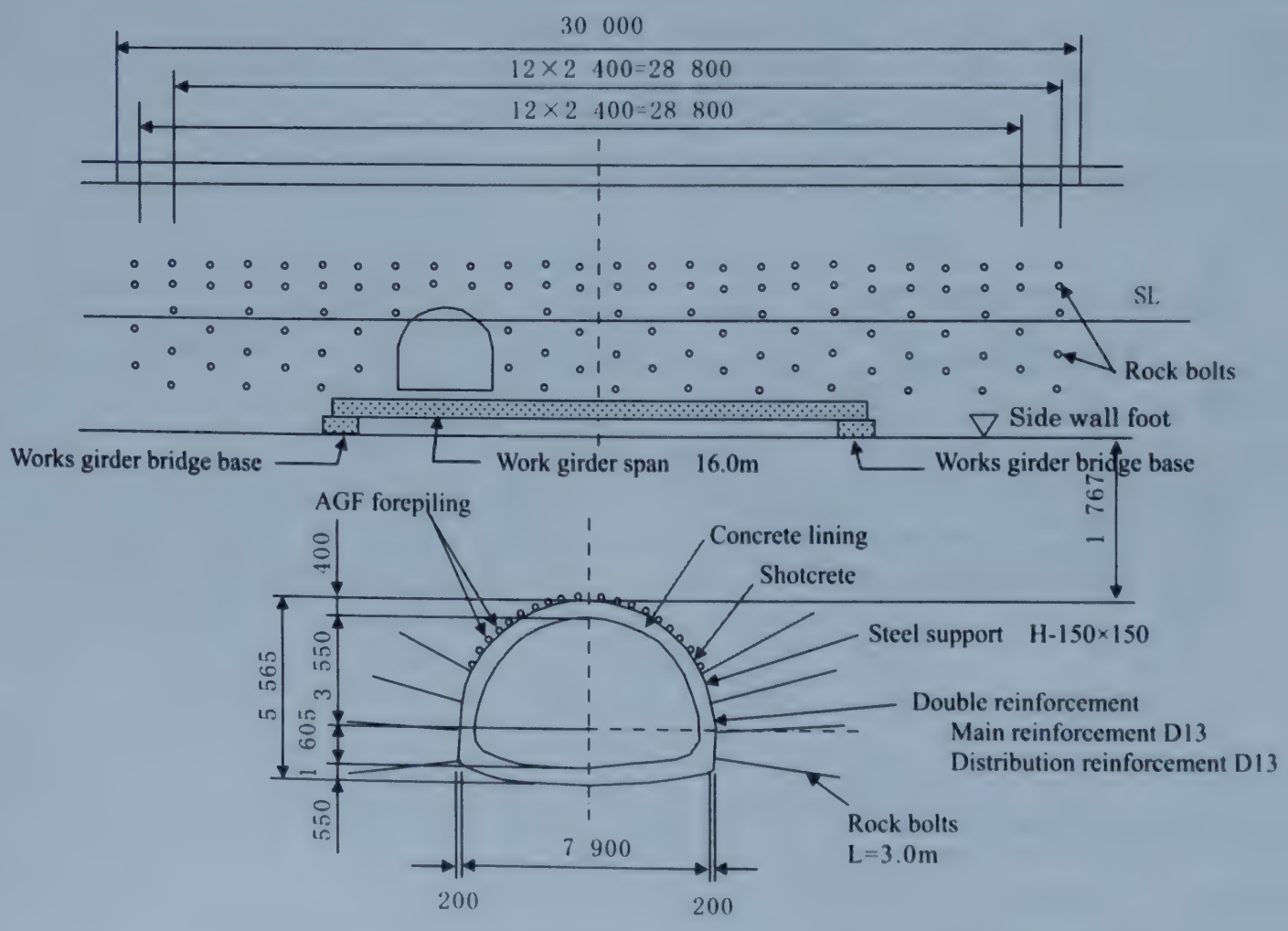


(a) Parallel tunnels (Minami-Osawa Tunnel)

Figure 4-9: Example of Parallel Tunnel



(b) Two-tiered parallel tunnel (Washuzan Tunnel)



(c) Crossing tunnel (Asamushi Dam Watershed Tunnel)

Figure 4-10: Example of Parallel Tunnel and Crossing Tunnel Section Shape (2)

iv) Control measures
Measures for the advance tunnel, measures for the second tunnel or measures for the ground between the tunnels must be properly selected and designed. Examples for each control measure are shown in Table 4-9.
In neighbouring tunnels, monitoring is especially important to verify the validity of the design and construction.

Table 4-9: Control Measures for Parallel and Crossing Tunnels

Where taken	Effect	Control Measures
Advance tunnel	Deformation	- Reinforcement of bearing capacity of supports, lining and inverts, early closure of a section
Second tunnel	Deformation	- Advance ground support, division of excavation section, early closure of section. - Support reinforcement, increase of bearing capacity of support foot, face reinforcement - Improvement of ground ahead of the face.
	Vibration	- Vibration control by means such as controlled blasting, mechanical excavation.
	Groundwater	- Drainage and water sealing
Ground between tunnels	Deformation	- Ground strengthening and improvement, interception using steel sheet piles, etc.
	Groundwater	- Drainage and water sealing

(2) Twin-Tube Tunnel
past construction records are useful for the design of a twin-tube tunnel. numerical analyses such as finite element analysis to analyse the behaviour of the surrounding ground is also useful. The suitability of the joint wall and supporting structure shall be studied for the design of a twin-tube tunnel.

i) Numbers of pilot tunnels
As shown in Figure 4-11, a twin-tube tunnel can be constructed by a single heading or triple heading method. Generally, the single heading system is adopted in the excellent ground and the triple heading system is adopted in the defective ground where the bearing capacity is considered inadequate.
Twin tunnels accompany the construction of the joint wall, but examples can also be found where the heading for the twin-tube tunnel is eliminated and the work carried out as twin tunnels by making appropriate reinforcement to the support and lining of the precedent tunnels. An example of the section shape of twin tunnels is shown in Figure 4-12.

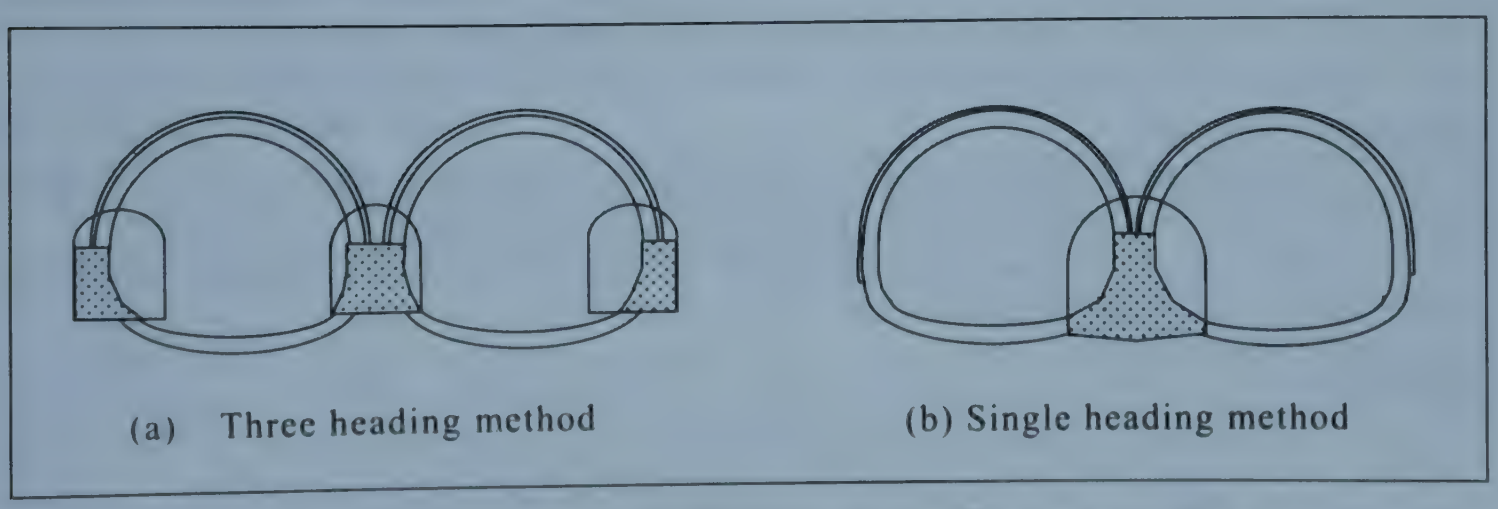


Figure 4-11: Classification of Twin-tube tunnel Excavation Method

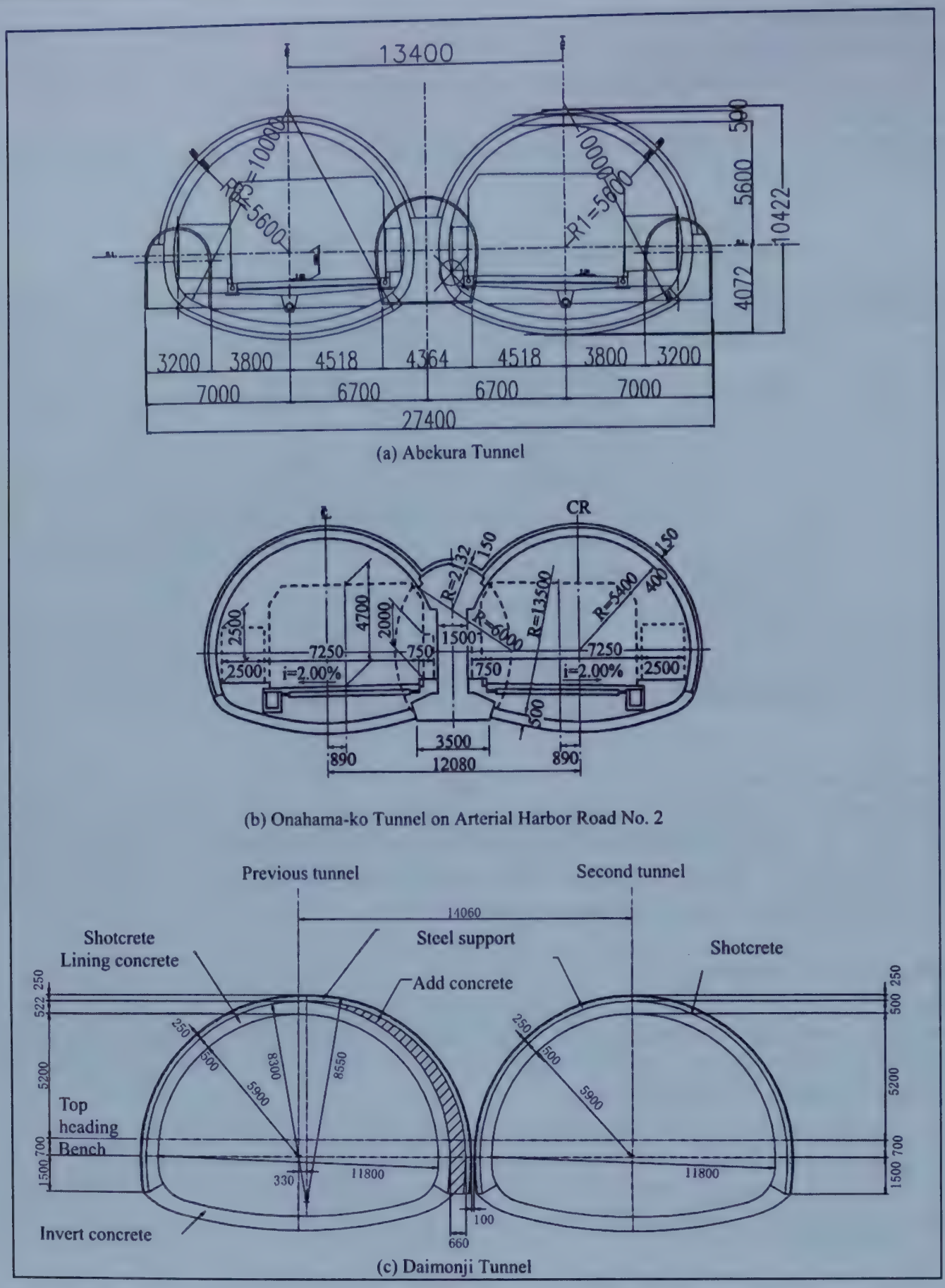


Figure 4-12: Example of Twin-tube tunnel

ii) Design of joint wall

The joint wall receives loads repeatedly from multiple directions when excavating both main tunnels, and must maintain the stability of the entire tunnel.

The joint wall section shape can be classified into a symmetric section type and asymmetric section type. The characteristics of each type are shown in Table 4-10.

The section shape shall be decided after sufficiently considering these characteristics.

As shown in Table 4-11, the support structure of the lining of the twin-tube tunnel can be classified into three types. For the selection of the lining support structure, the respective characteristics must be understood and the support structure must be designed by giving consideration to the site width, ground condition, drainage structure, and other factors.

Table 4-10: Joint Wall

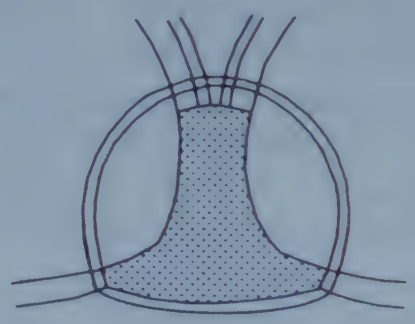
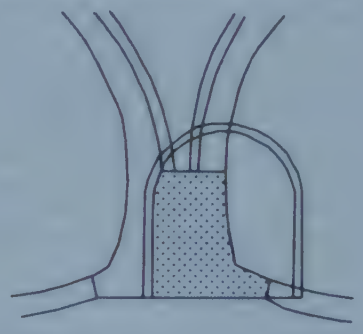
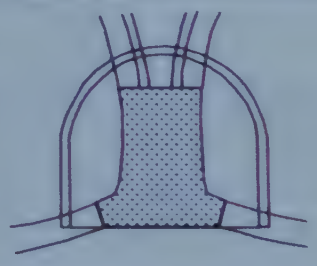
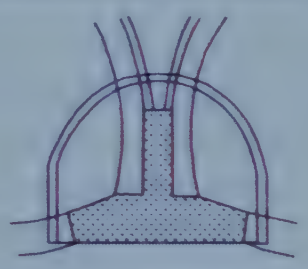
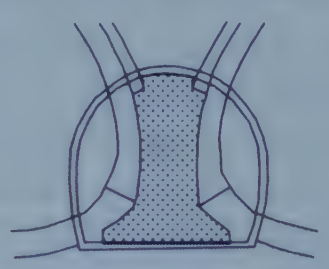
	Symmetric section type	Asymmetric section type
Section shape		
Characteristics	- Ease of construction and superior stability when tunnel is completed. - Depending on ground condition, stabilization measures required when excavating main tunnels because cavities are created on both sides of the center pillar.	- Heading section can be reduced. - Easily causes stress concentration and unequal settlement. - Concrete lining shape on the following tunnel side worsens.

Table 4-11: Lining Support Shape

	a	b	C
Section shape			
Characteristics	- Center diaphragm serves the role of lining. - Difficult to take control measures when deformation of center diaphragm occurs during construction. - Waterproofing and drainage structure are difficult.	- Center diaphragm serves the role of lining. - Difficult to take control measures when deformation of center diaphragm occurs during construction. - Waterproofing and drainage structure are superior to a type section	- Superior stability as a tunnel because lining and invert are combined. - Width of center diaphragm sole plate is small, and stability during construction can become a problem. - Excellent waterproofing and drainage structure.

iii) Support design

The supporting structure for the twin-tube tunnel shall be referred to the results of past construction, and the suitability shall be studied based on numerical analyses such as a finite element method analysis.

Compared with the supports for the single tunnel in the same ground conditions, supports for the twin-tube tunnel are reinforced more frequently. In many cases, the supports for the advanced tunnel and the second tunnel are the same.

iv) Lining of invert design

In some cases, the lining of the advanced tunnel is constructed before the second tunnel is excavated, and in some cases is constructed after the following tunnel has been excavated. Because the precedent tunnel lining can be affected by the following tunnelling works, the degree of this effect shall be estimated, and reinforcement works or other measures shall be studied.

Sufficient study is required because the inverted shape, structure, and construction period can greatly affect the stability of the joint wall and the entire tunnel.

v) Control measures
Because twin-tube tunnels frequently are utilized in urban areas where the overburden is small and the ground is soft, and the total excavation diameter is large, there are numerous examples in which methods such as fore-poling have been adopted to control subsidence and prevent sudden fall of the ground surface. The reinforcement works for the twin-tube tunnel shall be selected by referring to examples of similar construction and taking into consideration factors such as ground condition, location conditions, overburden, the excavation system, and the structure of the joint wall.

4.3.4 Affected by Construction Works in the Vicinity

(1) Effects from Construction Works in the Vicinity
the impact on the tunnel from the construction works in the vicinity must be estimated forehand, before the construction works in the vicinity can vary greatly depending on factors such as the scale of the construction works, overburden, ground condition, the tunnel structure, the tunnelling method, and tunnelling work sequence.
The impacts caused by the construction work in the vicinity are as follows.

i) Tunnel deformation and stress
The tunnel might suffer deformation on the side of the construction works, the ground surrounding the tunnel might loosen further, and the loads acting on supports and the lining might increase.

ii) Effects from lowering of groundwater
In some cases, the groundwater level may be lowered or increased by the change in the groundwater environment. When tunnelling works causes the water level to fall, this may lead to consolidation settlement or uneven settlement across a wide area. The tunnel may be negatively affected by an increase in groundwater pressure as a result of the water level rising.
Examples of tunnel deformation caused by the construction work near the tunnel are shown in Figure 4-13.

(2) Effect Prediction Methods
Because the main effect is an increase in stress on the tunnel, the structure design should be prepared by methods such as finite element analysis that takes the consideration works into the increase in earth pressure and water pressure.

(3) Control Measures
Control measures shall be a combination of i) control measures on the side of the construction works in the vicinity, ii) control measures for the ground between the construction work and the tunnel, and iii) control measures on the tunnel, which shall be studied and prepared reasonable design.
These design measures are studied and generally used at the same time as required.
A list of control measures on the side of the construction works in the vicinity is shown in Table 4-12. Advance control measures on the side of the construction works is shown in Figure 4-14; In neighbouring tunnels, monitoring is especially important from standpoint of design for the side closest to the construction works and confirming the appropriateness of the construction works. Together with promptly making an evaluation of the monitoring results and studying the suitability of the original design for the side closest to the neighbouring, if necessary a modified design must be implemented to reflect the evaluation in the construction works.

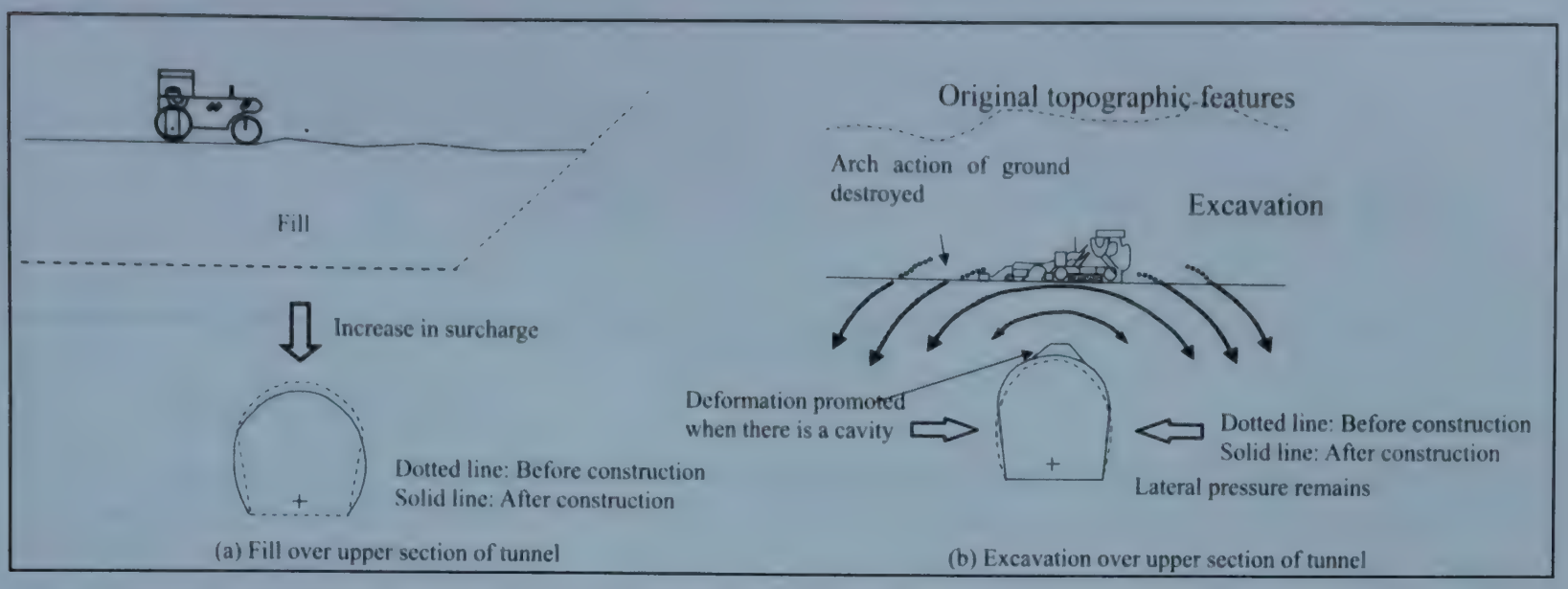

Fill
Increase in surcharge
Dotted line: Before construction
Solid line: After construction
(a) Fill over upper section of tunnel
Original topographic features
Arch action of ground destroyed
Excavation
Deformation promoted when there is a cavity
Dotted line: Before construction
Solid line: After construction
Lateral pressure remains
(b) Excavation over upper section of tunnel

Figure 4-13: Type of Impact on the Tunnel from Construction Works in the Vicinity

Table 4-12: List of Specific Methods to Control the Impact from Construction Works in the Vicinity

Type of works in the vicinity	Basic approach	Specific Method
Excavation over the upper section of the tunnel.	- Remove load evenly	- Revise excavation method - Revise excavation sequence - Limit depth of cutting
Fill over the upper section of the tunnel	- Cause even action of the load	- Revise spreading sequence - Revise spreading depth
Structure foundation over the upper section of tunnel	- Cause even action of the load - Limit excavation and concrete placement effects	- Revise foundation type - Revise method of works
Excavation on the tunnel side	- Remove load evenly - Limit displacement of the ground caused by excavation	- Revise excavation method - Revise excavation sequence - Revise earth retaining, slope gradient
Anchor near tunnel	- Eliminate immediate effects	- Revise anchor placement - Revise pre-stress work sequence
Flooding over the upper section of the tunnel	- limit effects from flooding	- Control the amount of flooding - Revise water cut-off
Ground vibrations	- Limit vibrations	- Utilize controlled blasting. - Study other construction methods. (Mechanical excavation, etc,.)

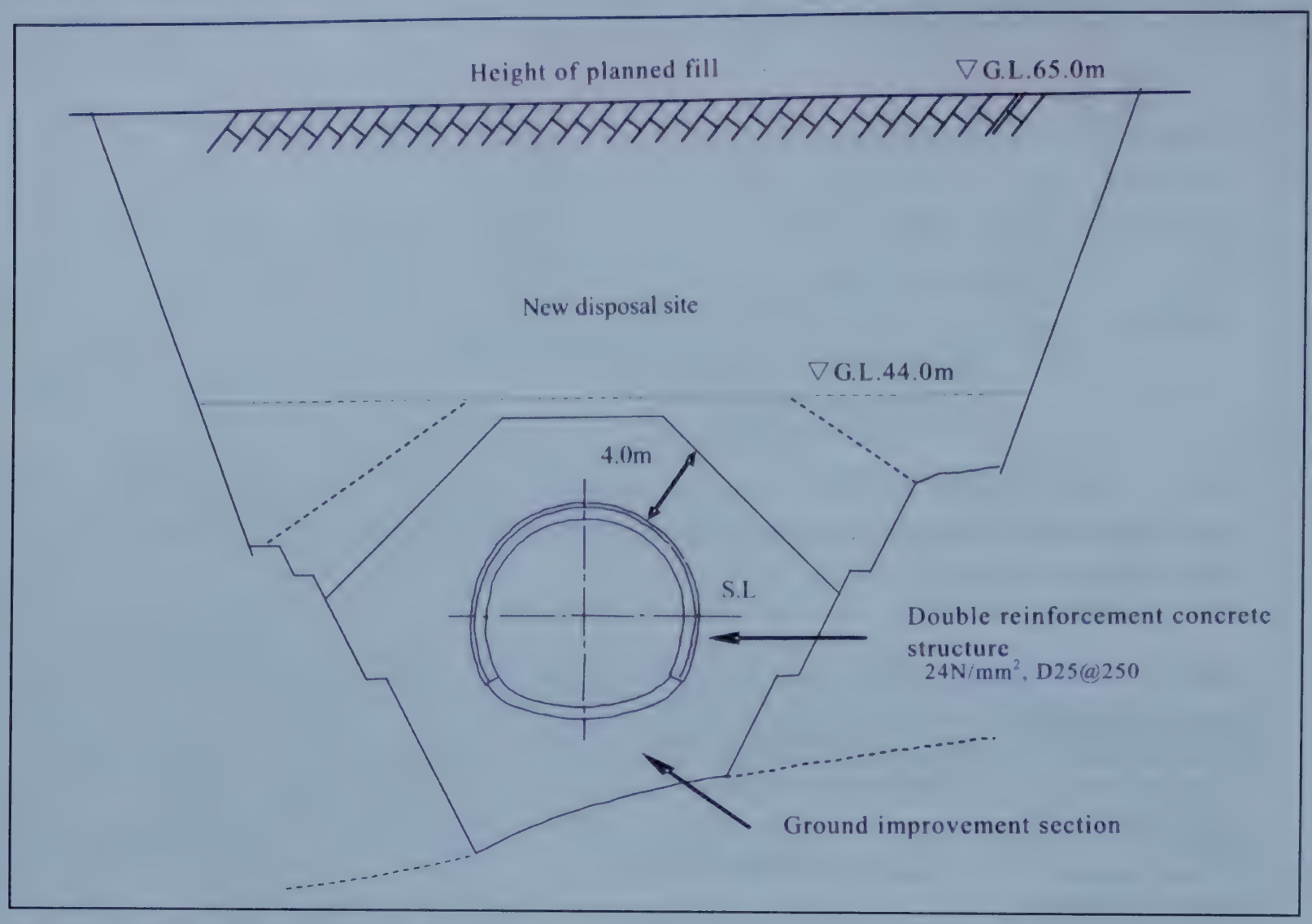


Figure 4-14: Example of Control Measures between Construction Site in the Vicinity and Tunnel

CHAPTER 5 WORK CONTROL

5.1 Quality Control

Rock conditions and actively utilizing the inherent strength and deformation of the rocks must be ensured to the stability of the tunnel. In order to manage the series of operations involved to achieve appropriate execution and reasonable cost-effectiveness, the behaviour of rocks and supports shall be monitored. If the design and/or construction are judged inadequate, proper correction shall be made as soon as possible.

Safekeeping of records concerning tunnelling works shall be made so that the records can be utilized for maintenance and repair. Reinforcement or other maintenance of the tunnel shall be kept in maintenance records. Examples of work records that are made during tunnelling works may include geological/geotechnical survey, quality control, tunnel monitoring/inspection, work records of tunnel supports and reinforcement works.

5.1.1 Schedule Control

The planning schedule and facilities are determined based on geological surveys, etc. conducted prior to the start of the works. Therefore, the planning does not necessarily coincide with the work schedule. The work schedule shall be analysed at every stage of the project to maintain control of the schedule in order to analyse the problems and then take appropriate action.

5.1.2 Shotcrete

(1) Materials, Weighing, and Mixing of Shotcrete

1) The items and method of the quality tests of materials for shotcrete shall be based upon the standard specification.

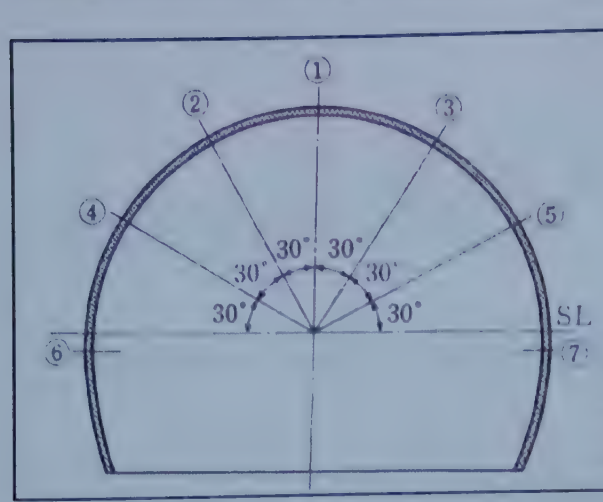
2) The amount of the shotcrete materials to be stored shall be determined, taking into consideration the work volume and schedule. Consumption estimate of the shotcrete materials is necessary since long-term storage can result in deterioration in the quality of materials. Materials shall be stored in a suitable installation to avoid intrusion of foreign substances.

3) Adequate control of mixing is important for the production of shotcrete. For this purpose, static and dynamic loading tests shall be carried out periodically to keep the accuracy of the scale and performance of the mixer. The abrasion progressive components of the mixer such as blades shall be inspected periodically, and shall be replaced or repaired as needed.

(2) Thickness and Strength of Shotcrete

1) Thickness of Shotcrete

Pins which can indicate the shotcrete thickness are recommended to apply shotcrete so that the design thickness may be achieved. For controlling the shotcrete thickness, it is recommended to follow a work control manual prescribing the inspection/monitoring interval, locations, and methods of inspection. An example of an inspection table of shotcrete thickness is shown in Figure 5-1.



Station			Design thickness					
Measuring point	1	2	3	4	5	6	7	Average
Measured thickness								

Figure 5-1: Example of Inspection Record of Shotcrete Thickness

2) Strength of Shotcrete

The quality and strength of shotcrete are affected by the skill of "Nozzleman", and by the rock conditions at the time of placement. Due to rebound, the mix proportion of shotcrete is usually different from that at the time of mixing. Shotcrete strength tests are classified into methods: the initial strength test method, early-stage strength, and long-term stage strength test method and the method for testing fibre-reinforced shotcrete, as described below:

a) Testing method to determine the initial strength
   - Pull-out test of pins, using specimens sprayed on a form with pins installed beforehand
   - Pull-out test of pins embedded at the time of shotcreting
   - Comprehensive test of specimen sprayed on a beam form
   - Penetration test of a pin driven into the shotcrete surface with air pressure

b) Testing method to determine the early stage strength and long-term stage strength
   - Comprehensive test of cores sampled from the specimens sprayed on a form
   - Compressive test of cores directly sampled from the specimens sprayed on the tunnel wall

c) Testing method to determine bending toughness of steel fibre-reinforced shotcrete
   - Bending toughness test of specimens sampled from shotcrete sprayed on a form

Since the purpose of strength tests is to confirm the strength of shotcrete sprayed on the rocks, it is recommendable to conduct the tests on site. While most tests are conducted on a basis of the 28-days strength, the initial strength is, particularly for shotcrete. Thus, effort should be made to inspect the initial strength by selecting a testing method available for specimens at an early stage.

5.1.3 Rock Bolts

(1) Material of Rock Bolts

1) Prior to the use of rock bolts including bearing plates and other materials, the quality, shape, dimensions of the materials shall be inspected or tested. The tests and inspections may not be necessary if an inspection certificate of the manufacturer is obtained. An example of typical quality tests and their frequency is given in Table 5-1.

2) Quality of the bonding material shall be inspected prior to use it. The inspection may not be necessary if an inspection certificate submitted by the manufacturer is obtained.

Table 5-1: Example of Quality Tests of Rock Bolt

Test item	Test Method	Test Frequency	Remarks
Appearance	Visual	- Once prior to the work - Every time a product is delivered, - Once when manufacturing factory or quality is changed	No harmful damage
Shapes / dimensions	Check dimensions		In accordance with relevant specification
Materials	Inspection certificate of manufacturing factory		In accordance with relevant specification

(2) Arrangement and Bonding of Rock Bolts

1) The rock bolts shall be placed in the arrangement (position and direction) and length specified in the design document. Should any change have been made, it is necessary to ensure by means of monitoring that the rock bolts demonstrate a working effect equivalent to the design effect. Examples of monitoring rock bolt hole drilling accuracy are given in Table 5-2.

Table 5-2: Example of Monitoring of Rock Bolt Hole

Monitoring item	Monitoring method	Monitoring frequency	Remarks
Position/number of holes	Visual	all holes	Mark the rock bolt position
Direction	Measurement	as necessary	Confirm with a grade scale
Hole diameter	Measurement	as necessary	Confirm by the bit diameter.
Length	Measurement	as necessary	Confirm by the rock bolt length.

2) In some cases, if the drilled hole has a rough hole wall or collapsed, the rock bolts are hard to be inserted and bonding material is hard to be filled. It is, therefore, necessary to confirm in advance that the required holes are available.

The pull-out strength is set at a level equivalent to the yield strength of the rock bolt (usually, yield strength of the threaded portion). When the manufacturing factory or quality is changed, the strength of materials shall be verified. In the pull-out test during construction, the rock bolts are judged to be adequate, if it exhibits about 80% of the stipulated pull-out strength. An example of an examination of rock bolt bonding is shown in Table 5-3.

Mortar is one of the bonding materials most widely used. Its mixing and weighting are managed. Examples of consistency and strength of pre-filling type mortar are shown in Table 5-4.

Table 5-3: Example of Inspection of Rock Bolt Bonding

Inspection item	Method	Frequency	Remarks
Bonding force	Pull-out test	Every 20 m in the initial stage of the excavation, then every 50m: about 3 bolts per section respectively (one in the crown, arch, side wall)	Judged to be adequate of about 80% of the pull-out strength set by the preliminary test is achieved. The later the testing, the more likely the predefined pull-out strength is to be exceeded, even if the filling of bonding materials is insufficient. It is therefore recommended that the time for the test be set on the basis of the age of the bonding material that exhibits a pull-out strength equivalent to the yield strength of rock bolts.
Filling of bonding material	Visual	Every time rock boil is installed	After inserting the rock bolt, make sure that the hole is entirely filled with the bonding material.

Table 5-4: Example of Examination of Bonding Material

Item	Method	Frequency
Consistency	Flow Test	- Once before the start of work - During tunnelling, or whenever necessary - Once when the manufacturing factory or quality is changed.
Strength	Compressive strength test	- Once before the start of work. - Once per 50 m during work execution. - Once when the manufacturing factory or quality is changed.

3) It should be verified that the bearing plate is in close contact with the surface of the shotcrete or excavation face. This inspection is usually done visually or by striking lightly with a hammer. Deformation or other abnormalities of the bearing plates shall be visually monitored until the placement of the lining is complete.

5.1.4 Steel Ribs

(1) Materials of Steel Ribs

The materials of the steel ribs shall be verified that they conform to the design referring to the inspection certificate of the manufacturer. The shapes, dimensions and processing methods of the steel ribs, such as bending, cutting, drilling, welding shall be verified that they conform to the design (including approved processing drawings).

(2) Erecting of Steel Ribs

To achieve sufficient supporting function, each set of steel ribs must be in the same plane, free from torsion and inclination (except for inclined shafts). In addition, it must be confirmed that there are no significant irregularities between adjacent steel ribs, that they are erected at a specified interval and position, and the correct number of supports of the correct shape is erected.

5.1.5 Lining

(1) Material Mixture and Strength of Lining

The material, mixture, and strength of lining concrete shall be controlled properly.

(2) Installation of Formwork and Inspection of Lining

It shall be confirmed that the shapes and dimensions of the formwork are correct and that the structure has sufficient strength for bearing the pressures of concrete. In addition, it shall be confirmed that the rocks below the formwork have a sufficient load capacity.
Prior to concrete placing, the formwork shall be checked to ensure that the specified shapes and dimensions of the lining have been obtained and that the designed lining thickness has been achieved.
The lining shall be inspected to ensure that it has the specified shapes and dimensions and its surface is in a satisfactory state.

5.1.6 Waterproofing and Crack Protection

It shall be confirmed that the quality of the materials used for waterproofing and crack protection shall be confirmed to satisfy the design requirements, and the prescribed inspections shall be conducted whenever necessary. In addition, the materials shall be properly stored in such a manner that the quality does not deteriorate.
When conducting waterproofing and crack protection, inspection shall be conducted to make sure that the substrate has been properly prepared, waterproofing performance and continuity of each material used are suitably maintained, and the materials are securely reinforced and fixed to avoid damage and falling concrete placement.

5.1.7 Drainage

It shall be confirmed that the quality of the materials used for drainage satisfies the design requirements, and the prescribed inspections shall be conducted whenever necessary.
It shall be confirmed that the construction of drainage conforms to the design requirements. In addition, it shall be confirmed that sufficient drainage capacity is achieved.

5.2 Instrumentation and Monitoring

5.2.1 Planning of Instrumentation and Monitoring

A monitoring plan shall be established on the basis of the following basic policies and planning items below.

1) Basic policies
   - Understanding problems such as the rock behaviour varying with the progress of excavation (clarifying the purpose of monitoring)
   - Determining items for monitoring (clarifying the monitoring objects)
   - Determining the control level and reinforcement measures to be taken (evaluation of monitoring results, use of the results)
2) Planning items
   - Selecting items, positions, and frequency of monitoring.
   - Selecting the equipment and instruments to be used.
   - Establishing the monitoring methods.
   - Selecting evaluation methods for monitoring results.
   - Selecting methods to utilize the findings in the design and tunnelling.
   - Determining the record formats.
   - Setting up the organization for information exchange.

(1) Purpose of Inspection and Monitoring

The purpose of monitoring is to verify the stability of the tunnel face, the safety of installed tunnel support systems, the stability of surrounding rocks and neighbouring structures during construction. At the same time, it aims to help identify unexpected factors so that the design may be modified according to the actual condition at tunnelling site in order to minimize the tunnelling cost.

It is necessary to quantitatively identify the actual behaviour of the ground surrounding the tunnel and the effect of the tunnel support system, on the basis of the results of monitoring during construction, and to modify the design and tunnelling methods according to pre-defined deformation criteria.

The purpose and roles of monitoring are shown in Figure 5-2.

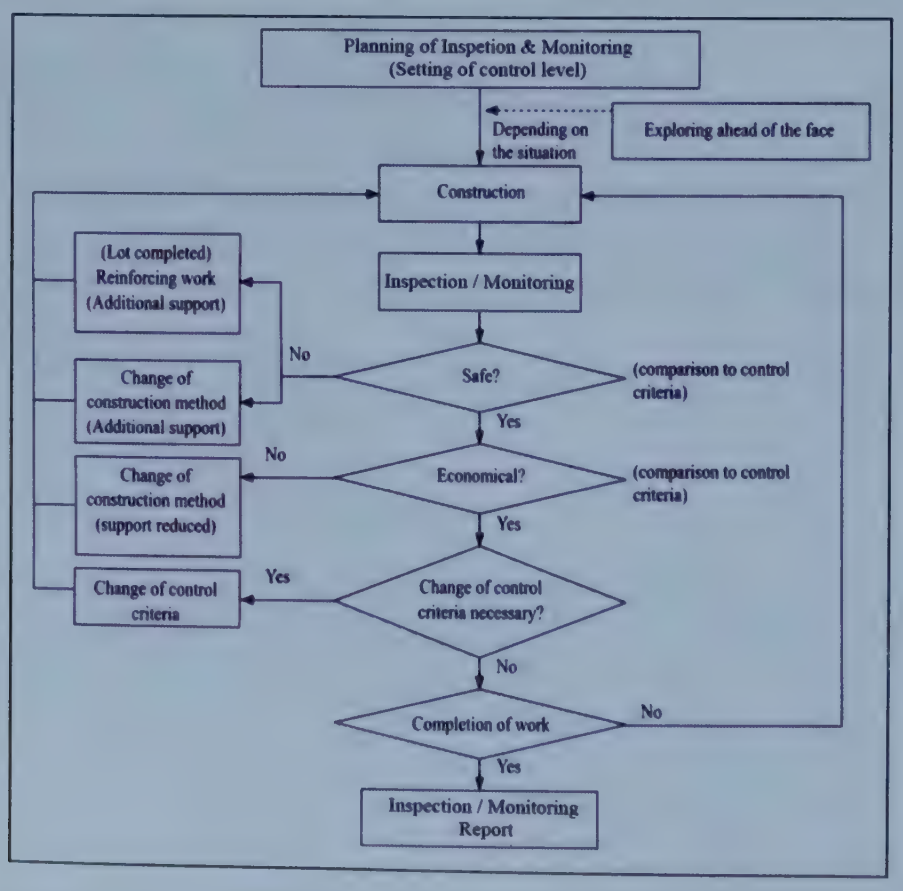


Figure 5-2: Purpose and Role of Inspection and Monitoring

(2) Monitoring Item

The items for monitoring of the tunnel and surrounding rocks are classified into the following two categories, each of which requires selection of necessary items according to site conditions:

i) Monitoring concerning the stability, behaviour, etc., of rocks

ii) Monitoring concerning the stability, functionality, etc., of supports.

Table 5-5 shows a list of items subject to monitoring concerning the rocks and supports. Table 5-6 shows items subject to monitoring to be selected according to respective rock conditions.

The monitoring items may be classified into two categories: items for daily monitoring during the construction (A-Monitoring), and items performed in addition to A-Monitoring (B-Monitoring). The classification that is usually employed is shown in the rightmost column of Table 5-5.

A-Monitoring is conducted at regular intervals in the longitudinal direction of the tunnel, selecting the items related to the rocks and tunnel behaviour, for the purpose of obtaining data to identify anything unusual in the behaviour of the rocks and tunnel support systems, and their stability.

B-Monitoring is made in typical sections, for the purpose of utilizing the measured results in the succeeding design and work steps. The monitoring is made by evaluation of the suitability of the actual support configuration, on the basis of the relationship between tunnel behaviour, rocks, and all tunnel support behaviour information obtained from A-Monitoring.

Table 5-5: Examples of Major Inspection and Monitoring on Rocks and Supports

Category	Purpose	Position	Item to be observed	Use of results	Monitoring
Visual inspection of rocks and supports	Visual inspection	Inside tunnel	- Rock conditions of tunnel face and - State of supports/lining / water inflow	- Evaluate stability of tunnel face - Review the rock mass classification - Study the relationship between rock condition and deformation - Predict the future rock condition and groundwater conditions. - Maintenance during service.	A
		Surface	- Condition of surface	- Study sphere of influence of the tunnel - Evaluate stability of the surrounding ground	A, B
Survey/test on rocks	Laboratory tests and in-situ tests	Inside-tunnel	- Laboratory tests for rock properties (collect rock samples in the tunnel)	- Review the rock mass classification - Study the strength of rocks and review the rock deformation. - Find swelling rocks - Evaluate tunnel face stability	B
		Inside-tunnel	- In-situ tests for rock properties	- Check detailed rock conditions - Review the rock mass classification - Predict the geology ahead of tunnel face - Study the strength of rocks and review the rock deformation.	B
Monitoring of Rocks	Convergence between tunnel walls	Inside-tunnel	- monitor the distance between tunnel walls - Displacement at monitoring points on tunnel walls.	- Study stability of surrounding rocks - Study the effect of tunnel support systems - Study stability of linings	A
	Tunnel crown settlement	Inside-tunnel	- Settlement of crown and side walls	- Studying the stability of ground around the crown	A
	Settlement at foot of tunnel walls	Inside-tunnel	- Settlement at the support foot	- Study of bearing capability of the foot	A
	Heaving	Inside-tunnel	- Status of heaving	- Study stability of rock around invert.	B
	Rock deformation around the tunnel	Inside-tunnel	- Radial displacement of the surrounding ground	- Confirm the rock plastified zone. - Review the lengths of rock bolts	B
	Ground deformation monitoring from the ground surface	Surface	- Settlement in the surrounding ground - Horizontal displacement in the surrounding ground	- Study ground behaviour around the tunnel - Study 3D deformation of the ground - Study stability of ground ahead of tunnel face	B
	Ground surface deformation	Surface	- Settlement - Landslide	- Study sphere of influence of the tunnel - Study stability of rocks ahead of tunnel face - Monitor landslide activity	A, B
Monitoring of tunnel support systems	Axial force of rock bolt.	Inside-tunnel	- Axial force of rock bolt.	- Review rock bolt length, number, position, installation method	B
	Stress in shotcrete	Inside-tunnel	- Stress in shotcrete - Working load	- Review shotcrete thickness, strength - Review distribution of stress between shotcrete and steel ribs	B
	Stress in steel ribs	Inside-tunnel	- Stress, the sectional force of steel ribs.	- Review suitability of dimensions - Review support spacing of steel rib - Review distribution of stress between steel ribs and shotcrete	B
	Stress in lining	Inside-tunnel	- Stress in lining concrete - Stress in reinforcement bars	- Review safety of concrete lining - Review design of linings and timing of installation - Monitoring long-term behaviour of lining	B
	Deformation of lining	Inside-tunnel	- Convergence between lining walls - Displacement at monitoring points	- Review stability of lining	B
Others	Deformation in a nearby structure	Ground surface	- Settlement of the structure - Inclination of the structure - Vibration induced by tunnelling works	- Evaluate the impact on nearby structures	B
	Ground-water table	Inside-tunnel* Ground Surface	- Groundwater table - Porewater pressure*	- Study measures against groundwater inflow. - Evaluate water recharge. - Evaluate water pressure acting on tunnel lining	B

Table 5-6: Selection of Key Items for Monitoring for Different Ground Conditions

Rock condition and classification		Phenomena	Survey / Tests													
			Visual inspection and Survey	Convergence	Crown settlement	Settlement at the foot	Heaving	Rock laboratory test	In-situ test	Ground surface displacement	Ground displacement	Axial stress on rocks bolt	Shotcrete stress	Steel rib stress	Lining stress
Hard rocks	Few cracks	Rock fall	+	+	+										
	Many cracks without clay	Rock fall Pressure of loosened – ground	+	+	+										
	Many cracks and fractured	Rock fall, Loosening pressure, Actual earth pressure, Face stability	+	+	+						+	+			
Soft rock	Large qu/P_v ratio	Rock fall	+	+	+										
	Small qu/P_v ratio	Loosening pressure, Actual earth pressure	+	+	+	+	+	+			+	+			
	Extremely small qu/P_v ratio	Loosening pressure, Actual earth pressure, Face stability	+	+	+	+	+	+	+		+	+	+	+	
Soil		Loosening pressure face stability	+	+	+	+		+		+	+				
Swelling rocks		Loosening pressure actual earth pressure	+	+	+	+	+	+	+		+	+	+	+	+

qu/P_v ratio: ratio of uniaxial compressive strength (qu) to overburden pressure(P_v)

Laboratory tests include unconfined compression tests, ultrasonic tests, etc., and in-situ surveys and tests include in-tunnel seismic survey, jack tests, etc. shall be selected to suit the site conditions such as the rock conditions and survey purpose.

5.2.2 Monitoring

(1) Monitoring Method

1) Face monitoring

Face monitoring shall cover the following items:

i. Geology: Geological features and structures such as trikes and dips of strata
ii. Hardness/softness of rocks: Degree of weathering and decomposition as well as rock strength
iii. Cracks/fissures: orientation, spacing, condition, and properties
iv. Faults: positions, strikes, and dips of faults, degree of fracturing
v. Water inflow: location and volume of inflow, turbidity
vi. Face stability: spalling, degree of spalling, if any, the collapse of the face

All records should include objective comments on respective items and should be accompanied by sketches or photographs of the face. Records of all problems should include such details as the degree of severity of the problem condition or phenomena.

2) Monitoring of tunnel support systems

During tunnelling, the tunnel support systems should be surveyed with respect to the following:

i. Shotcrete
   - Cracks (location, type, width, length, and the rate of growth)
   - Water inflow (location, condition, volume)
ii. Rock bolts
   - Deformation of rock bolt or bearing plate
iii. Steel ribs

- Deformation and buckling (location, condition)
- Sinking into ground and foot settlement

iv. Lining
- Cracks (location, type, width, length, and rate of growth)
- Water leaks (location, condition, volume)

The survey and monitoring results should be recorded with details concerning the location, deformation type, volume of deformation, mechanism, etc., and should be accompanied by location maps and/or sketches, as needed.
Visual inspection of ground surface anomalies shall be made carefully during the survey and monitoring from the ground surface,

i) Ground surface deformation: cracks/fissures and deformation (time and location of occurrence, width, length, and the rate of growth)
ii) Vegetation: damage to trees
iii) Natural drainage systems: changes in springs and surface waters (volume, turbidity)
iv) Condition of neighbouring structures: condition of cracks (time and location of occurrence, width, depth, length, steps, and rate of growth)

(2) Layout of Monitoring

The following points are essential in the selection of the monitoring positions.

1) Tunnel face mapping
The exposed face conditions are documented in cross-section sketches (face mapping) drawn at frequent intervals as the tunnel advances over the entire length of the tunnel. In addition, when there is a problem such as a landslide near the portal, or when the overburden is not more than 2D (D=tunnel diameter), or when tunnel excavation may have an adverse impact upon an important structure on the surface, the ground surface survey is documented.

2) Monitoring of convergence and crown settlement
Convergence and crown settlement shall be measured in the same cross-sections. The monitoring section shall be designated at specified intervals, considering the ground condition and construction steps. Table 5-7 shows the typical monitoring intervals of convergence and crown settlement. The Layout of convergence monitoring is set out with consideration for the excavation method and foreseeable ground behaviour. Figure 5-3 shows examples of measuring lines.

Table 5-7: Typical Monitoring Intervals of Crown Settlement and Convergence

Rock mass classification	Within 50 m from the portal	2D or less of overburden depth (D=tunnel diameter)	The initial stage of construction (up to 200 m from the portal)	Regular monitoring
B, C	10 m	10 m	20 m	30 m (50 m in stable rocks)
D	10 m	10 m	20 m	20 m (30 m in stable rocks)
E	10 m	10 m	10 m	10 m

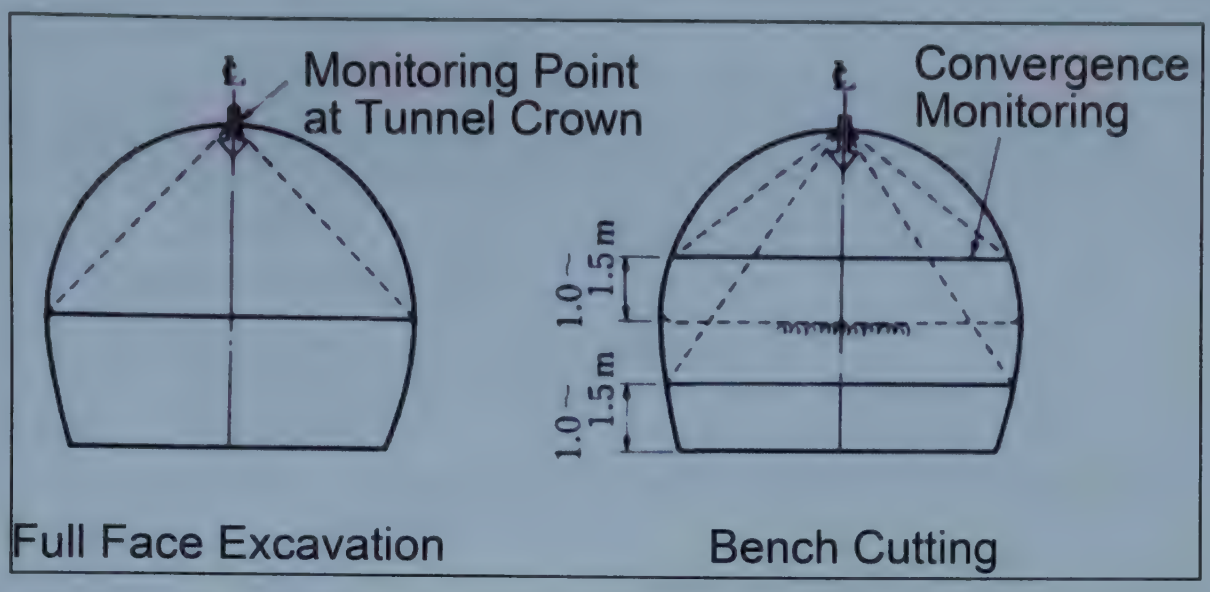


Figure 5-3: Examples of Monitoring Point and Lines of Crown Settlement / Convergence
(Tunnel Diameter is 10m)

3) B-Monitoring inside the tunnel

The monitoring items are tunnel deformation, axial stress of rock bolt, the stress in steel rib, shotcrete, and lining concrete. Since the primary purpose of these monitoring is to provide reference data for design and construction, they should be made in the initial construction stage, if practicable, selecting a sector where the ground conditions are the most representative of the project. Examples of arrangements of the main measuring instruments are shown in Figure 5-4.

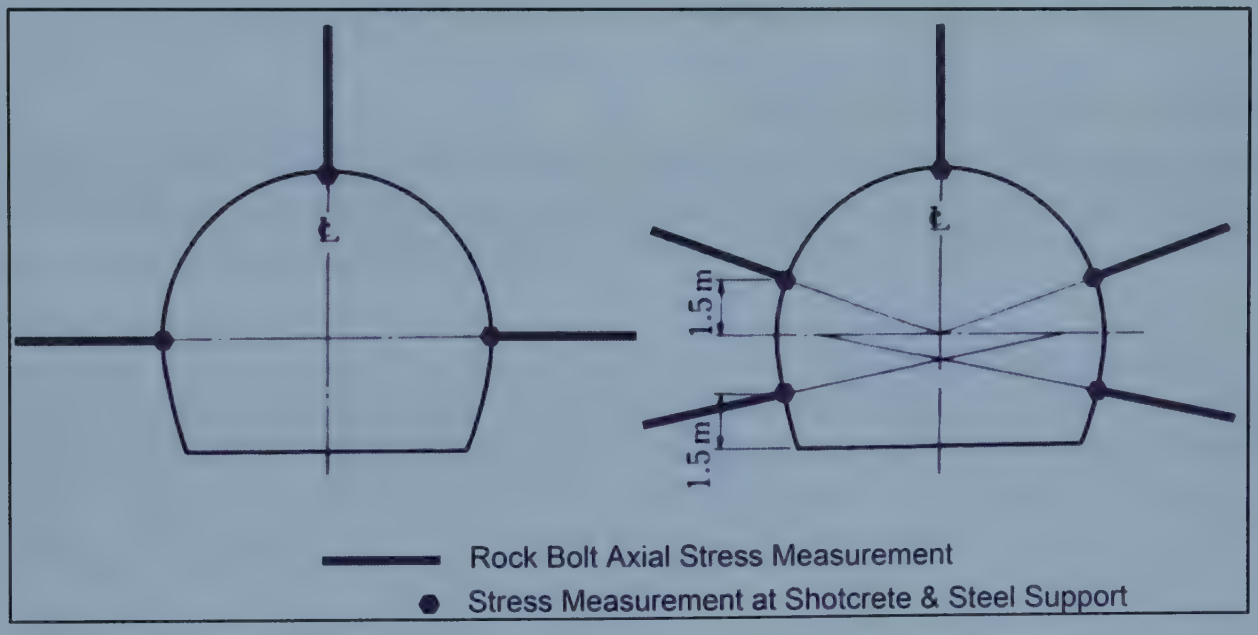


Figure 5-4: Examples of Arrangement of Rock Bolt Monitoring
(Tunnel Diameter is 10m)

4) Monitoring of surface and ground surface displacements

The monitoring from the surface is based on the guidelines provided in Table 5-8 according to the overburden. The interval for surface settlement monitoring shall be 5 to 10 m longitudinally, and 3 to 5 m in the cross-section, reducing the interval toward the centre of the tunnel, and also for lesser overburden. The measuring range in the cross-section is given in Figure 5-5, which corresponds to the range affected by excavation.

Table 5-8: Guidelines for Ground Surface Monitoring

Overburden	Importance	Necessity
H<D	Very important	Necessary
D<h<2D	Important	Recommendable
h>2D	Less important	If necessary

D: Tunnel diameter
h: Overburden depth

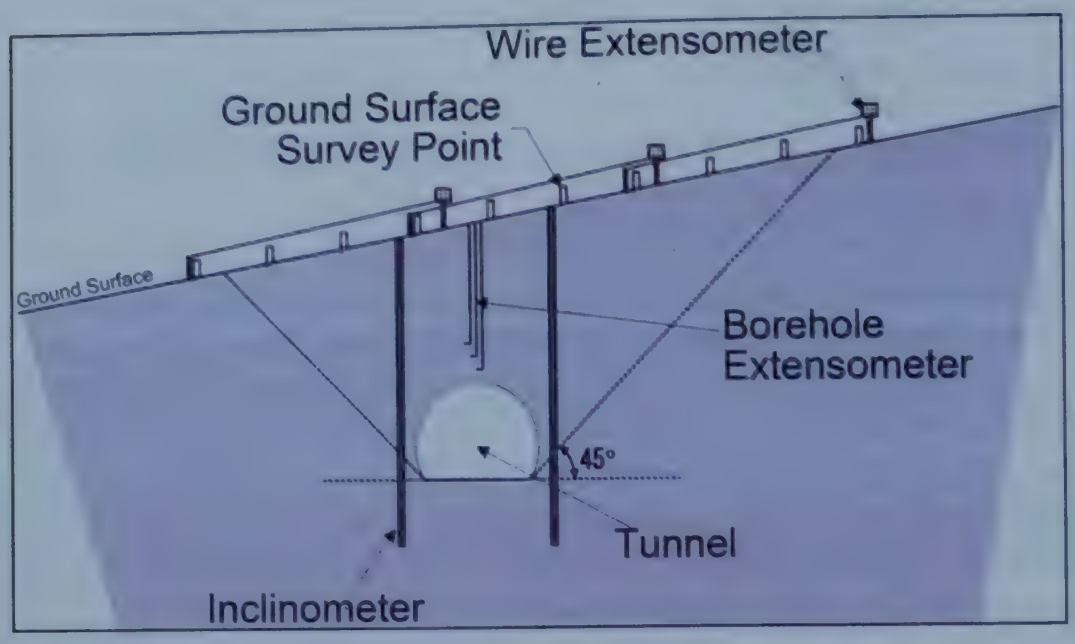


Figure 5-5: Example of Ground Surface Monitoring

5) Monitoring of groundwater table
Should a drop of groundwater table accompanying tunnel excavation be an issue, the groundwater table shall be monitored. Existing wells on the ground as well as water standpipes are useful for groundwater table monitoring.

(3) Frequency of Monitoring

As shown in Figure 5-6, the large displacement of the ground around the tunnel is seen 1D before and 1D after the passage of the face (D=tunnel diameter). The displacement of the ground converges at a certain distance from the face. The monitoring of tunnel deformation shall be frequently just before and after the passage of the face, and be a certain interval in some distance from the face. The monitoring frequency is set higher just before and after the passage of the face, and less frequent as the face is farther away. It is necessary to measure the initial value near the face just after excavation.

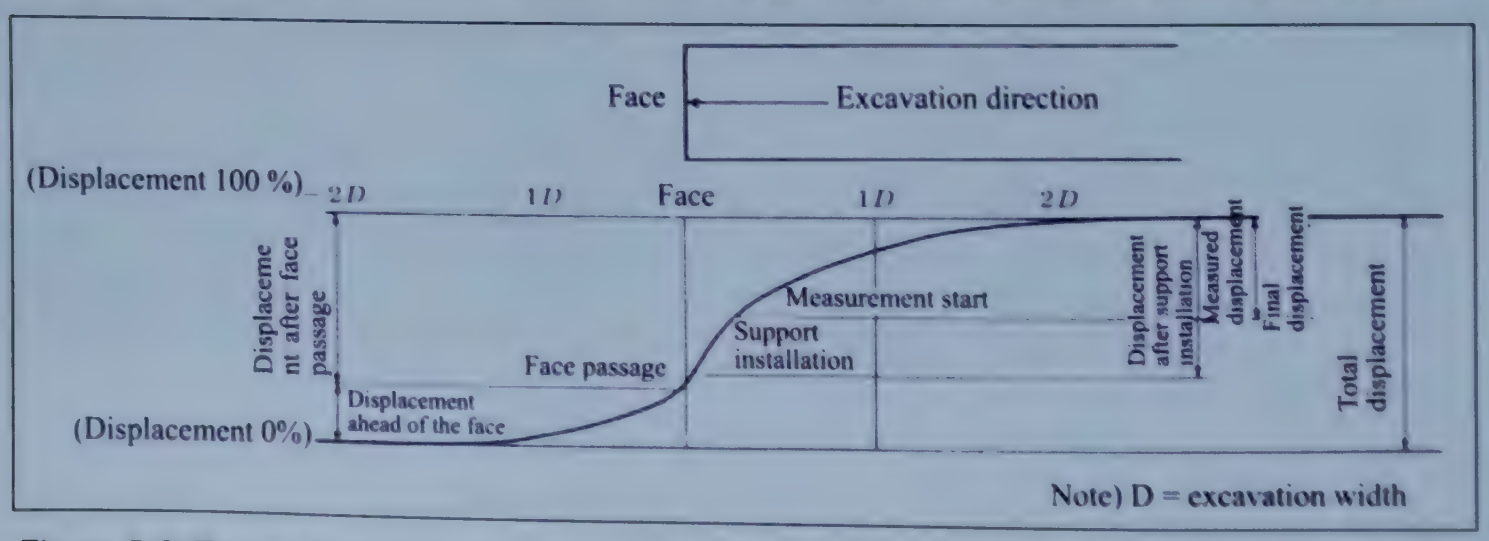


Figure 5-6: Typical Relationship between Face Position and Deformation of Surrounding Rocks

1) Monitoring
Geologic face mapping shall be kept daily in accordance with the progress of the face to be integrated into geological plans and longitudinal diagrams. Excavated sections also shall be surveyed daily. It is important to appropriately change the monitoring frequency, for example, to increase the frequency of the monitoring according to unexpected behaviour of the tunnel.

2) Monitoring of convergence/crown settlement
The frequency of monitoring of convergence/crown settlement shall be determined on the basis of distance from the face and displacement speed. Examples of monitoring frequencies of convergence/crown settlement of road tunnels are given in Table 5-9.

Table 5-9: Examples of Frequencies of Convergence and Crown Settlement Monitoring

Frequency	Distance of measuring point from tunnel face	Critical level
Twice / day	0-0.5 D	10 mm or more/day
Once / day	0.5-2 D	5-10 mm/day
Once / 2 days	2-5 D	1-5 mm/day
Once / week	5D or more	Less than 1 mm/day

- D = Tunnel diameter
- Monitoring frequency is determined by the rate of displacement, or by the distance from the tunnel face, whichever is the higher
- Deformation is under 1 mm/week for two consecutive weeks, monitoring can be terminated.
- The term "tunnel face" here refers to an excavation location such as the top heading, bench, invert, etc.

3) B-Monitoring inside the tunnel
Ground displacement, an axial force for rock bolts, stress for steel ribs, shotcrete stress, and lining stress shall be measured concurrently and on the same section as convergence/crown settlement monitoring.

4) B-Monitoring outside of the tunnel
It is important to start monitoring surface settlement, ground displacement, and other items outside the tunnel before the effects of excavation appear. B-Monitoring is generally made in the zone affected by excavation as shown in Figure 5-7, and the monitoring frequency may vary from once per day to once per week until displacement converges.

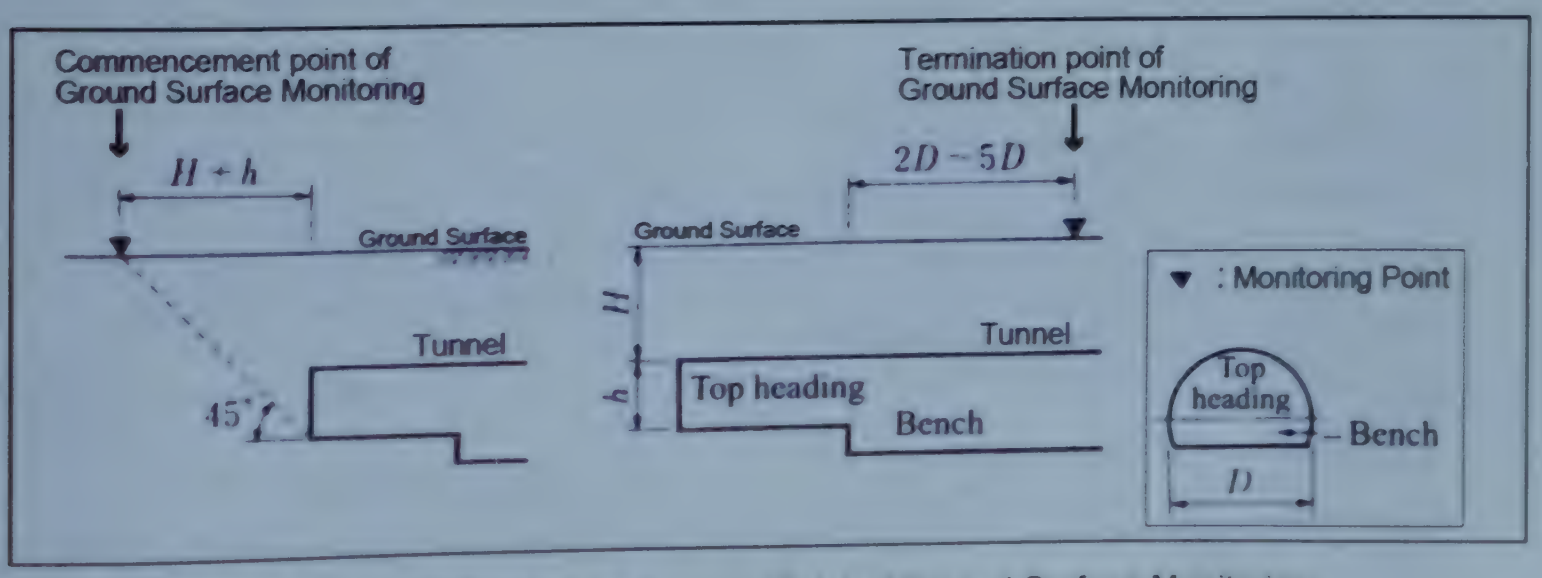


Figure 5-7: Commencement and Terminate Point of Ground Surface Monitoring
H: overburden depth, h: tunnel height, D: tunnel diameter

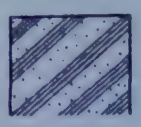
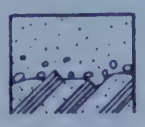
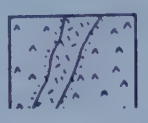
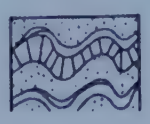
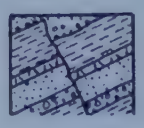
(4) Processing of Monitoring Results

All monitoring results shall be processed in a timely manner so as to determine the present condition of the tunnel and to utilize the findings in subsequent predictions and in the design and tunnelling.

Table 5-10a: Examples of Face Mapping Records

Name of tunnel		Location	Distance from starting point	
			Distance from portal	
Overburden	m	Comprehensive evaluation	Evaluation of ground classification or pattern classification	
Type of rock			Name of rock and geological age	
Status of special conditions	Swelling pressure, unsymmetrical pressure, fluidity, small overburden depth (m), important neighboring structures, directly under valley, others ( )			

Mode of occurrence

1. Alternation  2. Unconformity  3. Intrusion  4. Microfolding  5. Fault  6. Others

Condition and behavior of ground at excavation site

A	Face condition	1. Stable	2. Rocks fall off the face	3. Squeezing occurs at the face	4. Unable to support itself, the face collapses or flows out	5. Others
B	Condition of unsupported excavation surface	1.Standing (no support needed)	2. Loosening of ground and rock falls occur from time to time (ordinary support)	3. Barely able to stand and requires early support after excavation (immediate support)	4. Support before excavation needed	5. Others
C	Compressive strength	1. $\sigma_c \geq 100$ MPa Withstands hammer blows	2. $100 > \sigma_c \geq 20$ Crumbles when hit by a hammer	3. $20 > \sigma_c \geq 5$ Crumbles when hit lightly by a hammer	4. 5MPa$>\sigma_c$ A hammer head ends up lodged in	5. Others
D	Weathering/ alteration	1. No weathering; sound condition	2. Discoloring along rifts; slightly reduced in strength	3. General discoloration, considerably reduced in strength	4. Soil, clay, fractured, unconsolidated	5. Others
E	Crack/ fissure frequency	1. Spacing $d \geq 1$m	2. 1 m$> d \geq$20 cm	3. 20 cm$> d \geq$5 cm	4. 5 cm$>d$ Fracture unconsolidated	5. Others
F	Crack/ fissure conditions	1. Tight	2. Partially open	3. Open	4. Interbeds of clay, unconsolidated	5. Others
G	Mode of cracking/ fissuring	1. Random squares	2. Columns	3. Layers, fragments, plates	4. Soil, fragmented, unconsolidated	5. Others
H	Water inflow	1. None, seepage	2. Dripping	3. Concentrated	4. Entire face	5. Others
I	Deterioration due to water	1. None	2. Loosening	3. Softening	4. Collapse, outflow	5. Others

Crack/fissure orientation (presence of a discontinuous cutting face)

Longitudinal direction (cutting face)	1. Horizontal ($10° > \theta > 0°$) 2. Dipping into slope ($30° > \theta \geq 10°$, $80° > \theta \geq 60°$) 3. Dipping into slope ($60° > \theta \geq 30°$) 4. Dipping out of slope ($60° > \theta \geq 30°$) 5. Dipping out of slope ($30° > \theta \geq 10°$, $80° > \theta \geq 60°$) 6. Vertical ($\theta \geq 80°$) [in terms of maximum angle of slope]
Transverse direction (cutting face)	1. Horizontal ($10° > \theta > 0°$) 2. Right on left ($30° > \theta \geq 10°$, $80° > \theta \geq 60°$) 3. Right to left ($60° > \theta \geq 30°$) 4. Left to right ($60° > \theta \geq 30°$) 5. Left to right ($30° > \theta \geq 10°$, $80° > \theta \geq 60°$) 6. Vertical ($\theta \geq 80°$) [in terms of apparent angle of slope of cutting face]

(Japan Railway Construction Public Corporation, NATM Design and Construction Guidelines, February 1996)

Table 5-10b: Examples of Face Mapping Report in Railway Tunnel (Part 2)

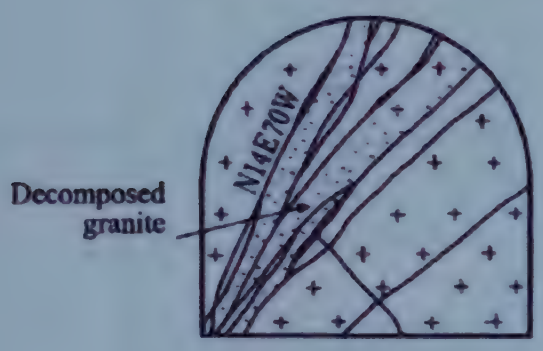
Provide additional information about the following items for unconsolidated ground,:

Ground conditions	Stratum conditions	1. Single soil layer 2. Alternation (1. Horizontal 2. Inclined) 3. Lens (1. No 2. Yes) 4. Others ( )				
	Special conditions	1. Talus layer 2. Mudflow layer 3. Boundary with rock mass 4. Cross section overlain by soft layer 5. Fill or embankment 6. Others ( )				
	Discontinuity	1. Well-developed cracks or fissures 2. Seam 3. Fault 4. Others ( )				
Soil type		1. Cohesive soil 2. Sandy soil 3. Gravelly soil 4. Special soil (1. Decomposed granite 2. Volcanic ash 3. Shirasu 4. Organic soil) 5. Others ( )				
Soil conditions (for special soil, select most applicable items.)	Cohesive soil	1. Soft ($4 > N$) 2. Medium ($8 > N \geq 4$) 3. Hard ($15 > N \geq 8$) 4. Very dense ($30 > N \geq 15$) 5. Consolidated ($N \geq 30$)				
	Sandy soil	1. Loose ($10 > N$) 2. Medium ($30 > N \geq 10$) 3. Dense ($50 > N \geq 30$) 4. Very dense ($N \geq 50$)				
	Gravelly soil	1. Loose 2. Compacted	Gravel size	1. 2-5cm 2. 5-20 3. 20-75 4. 75-300 5. 300cm ≤	Gravel content	1. ≤ 30% 2. 30–50% 3. 50% ≤
Ground properties		SPT blowcount (N-value)	Per-meability	1. Impervious layer 2. Impervious to relatively impermeable 3. Alternation of both 4. Others ( )		
Groundwater head		m above FL		Remarks		

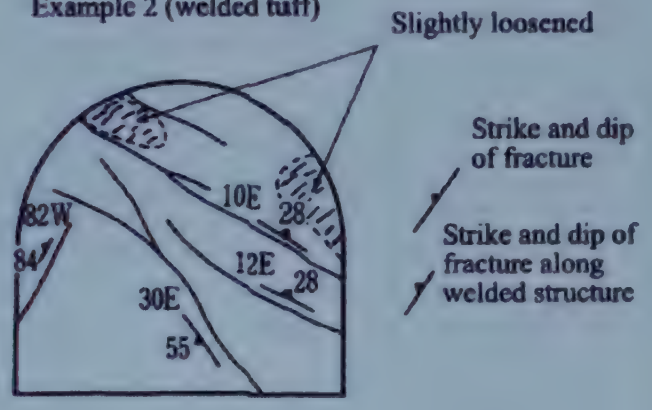
Sketch

Sketch of cutting face

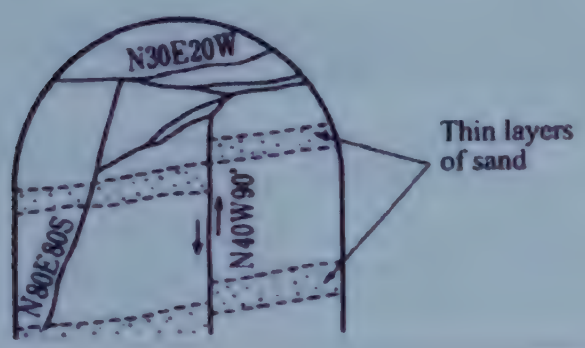
Example 1 (granite)



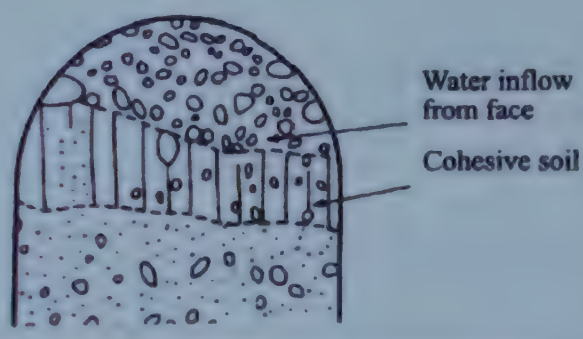
Example 2 (welded tuff)



Example 3 (Tertiary mudstone)



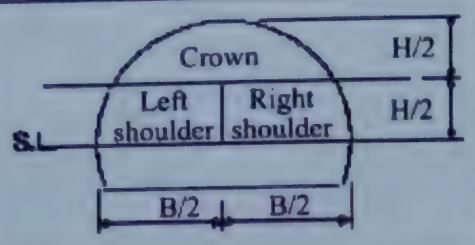
Example 4 (Quaternary terrace)



Name of observer	

Table 5-11: Examples of Face Mapping Report in Road Tunnel

Tunnel name		Observation date:	
Measuring station +	Distance from portal: m	Cross section: No.	Supporting pattern:
Overburden: m	Rock name/Geological age:	Rock group (1-5):	Rock code:
Specifications of auxiliary method (inc. face shotcrete and bolts)	Specifications of additional supports	Measurement A / B Specify A or B if it is the closest cross section	



Special conditions, status, etc. _____

_____

Presence/absence and status of collapse _____

Presence/absence of early invert closure _____

Observation item		Rating						Enter the ratings here		
								Left	Center	Right
A. Compressive strength (N/mm^2)	Unconfined compressive strength	100 or more	100 - 50	50 - 25	25 - 10	10 - 3	3 or less			
	Point load	4 or more	4 - 2	2 - 1	1 – 0.4	0.4 or less				
	Rough guide for strength in hammering	Hard to break when placed on the ground and strongly hammered	Breakable when placed on the ground and strongly hammered	Breakable when held by hand and hammered	Breakable when hit with another rock of the same type	Partially breakable using two hands	Crushable when force is applied with a fingertip			
	Rating	1	2	3	4	5	6			
B. Weathering and alteration	Rough guide for weathering	Generally fresh	Weathered or altered along the crack		Weathered or altered to the rock core		Weathered into soil or unconsolidated soil			
	Rough guide for hydrothermal alteration	No alteration observed	Clay observed in the crack due to alteration		Decreased strength observed at the rock core due to alteration		Remarkable alteration into soil or clay			
	Rating	1	2		3		4			
C. Crack intervals	Crack intervals	d ≥ 1 m	1 m > d ≥ 50 cm	50 cm > d ≥ 20 cm	20 cm > d ≥ 5 cm	5 cm > d				
	RQD	80 or more	80 - 50	60 - 30	40 - 10	20 or less				
	Rating	1	2	3	4	5				
D. Crack conditions	Crack opening	Tightly closed	Some of the cracks open (less than 1 mm wide)	Many of the cracks open (less than 1 mm wide)	Open (1-5 mm wide)	Open to the width of 5mm or more				
	Substance in cracks	None	None	None	Thin clay (5 mm or less)	Thick clay (5 mm or more)				
	Crack roughness and slickenside	Rough	Smooth	A limited slickenside	A highly polished slickenside					
	Rating	1	2	3	4	5				
E. Strike and dip	Strike perpendicular to tunnel axis	1. Reverse dip of 45 to 90 degrees	2. Reverse dip of 20 to 45 degrees	3. Regular/reverse dip of 0 to 20 degrees	4. Regular dip of 20 to 45 degrees	5. Regular dip of 45 to 90 degrees				
	Parallel with tunnel axis			1. Dip of 0 to 20 degrees	2. Dip of 20 to 45 degrees	3. Dip of 45 to 90 degrees				

Evaluation by water inflow and water-induced deterioration within 10 m from the face
(Deterioration shall be evaluated considering existing and possible future deterioration.)

F. Water inflow	Status	No inflow or water seeping not exceeding 1 liter/min.	Water dripping at 1 to 20 liters/min	Water locally inflowing at 20 to 100 liters/min.	Water extensively inflowing at 100 liters or more per min.				
	Rating	1	2	3	4				
G. Water-induced deterioration	Water-induced deterioration	None	Loosened	Weakened	Washed out				
	Rating	1	2	3	4				

5.2.3 Feedback of Monitoring

(1) Fundamental Approach

Monitoring results shall be promptly utilized for reasonable design and construction suitable for rock conditions. To this end, the final deformation of the surrounding rocks and the tunnel supports shall be analysed, and then the safety of construction, validity of the design, and possible impact on neighbouring structures shall be evaluated in accordance with the pre-defined control level.

The following list gives typical approaches for deciding control level to be based on:

i) Similar tunnelling cases in the past or records

ii) FEM analysis results

iii) Critical strain method

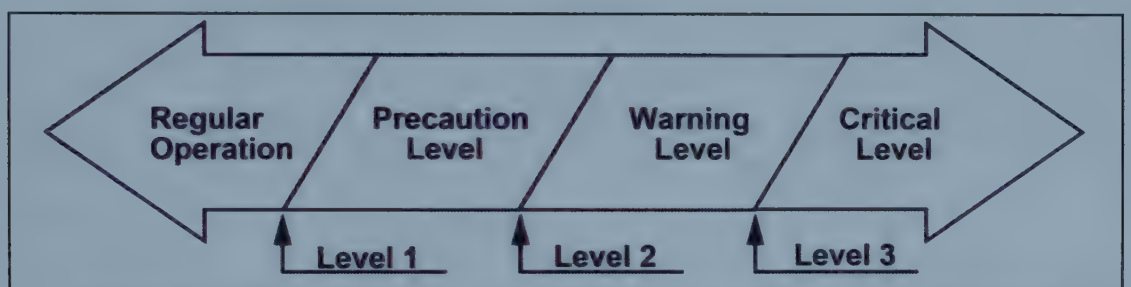
iv) Shearing index and shearing strain

v) Analysed final displacement

vi) The risk level for neighbouring structures

vii) Combination of approaches i) through vi)

A general relationship between the control level and the safety control system is shown in Figure 5-8.



Control Level	Action
Regular Operation	Standard Monitoring
Precaution Level	Increase the monitoring frequency Conduct site inspections Give strict instruction to workers
Warning Level	Strengthen the monitoring Commence reinforcement works
Critical Level	Suspend excavation works Analyse cause and tendency of deformation Revise the tunnel support design

Figure 5-8: Relationship between Control Level and Actions

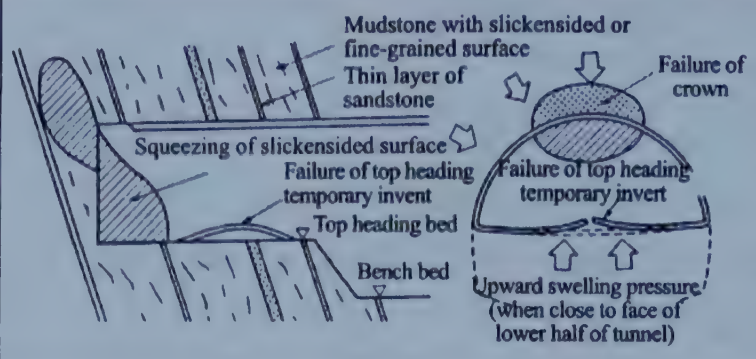
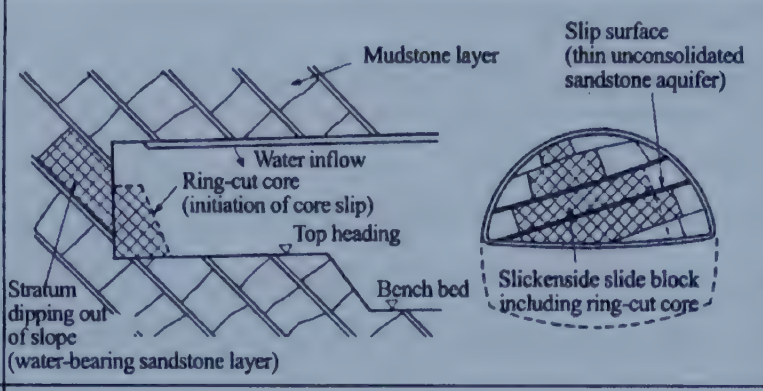
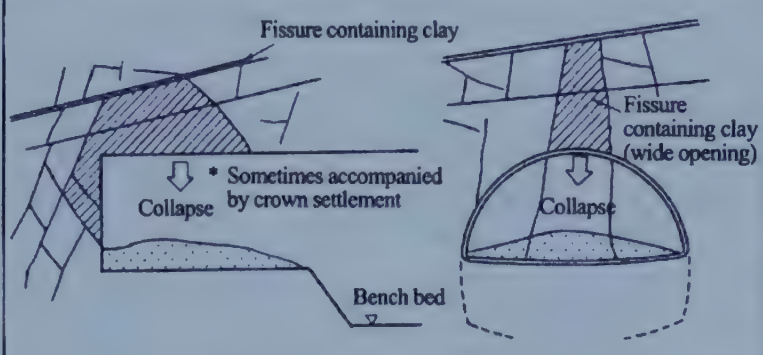
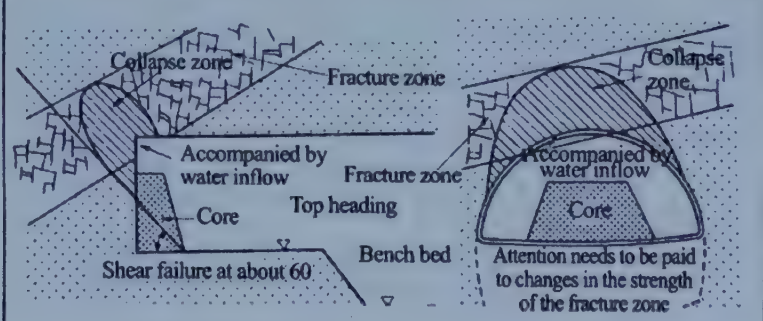
(2) Evaluation of Monitoring Results

1) Monitoring results

Monitoring and survey activities include face monitoring and monitoring of complete sections conducted inside the tunnel and surface monitoring conducted outside the tunnel. Evaluation methods for face monitoring results are classified into qualitative evaluation method and the quantitative evaluation method; the former comprehensively evaluates the monitoring results based on criteria established empirically, and the latter involves evaluation of rocks by aggregated scores that are converted from monitoring results.

Table 5-12 shows some models illustrating typical patterns of tunnel face collapse and lists important considerations in face monitoring.

Table 5-12: Mechanism of Cutting Face Collapse and Important Considerations in Face Mapping

Ground conditions		Typical face collapse mechanisms	Observation items	
(i) Uniformity	Largely uniform ground, such as expansive ground composed of Tertiary mudstone		Properties of bedrock	◎
			Degree and orientation of fissuring /cracking	△
			Water inflow	△
(ii) Bedding	Ground composed of alternating strata with different properties, such as sandstone or mudstone		Properties of bedrock	○
			Degree and orientation of fissuring /cracking	◎
			Water inflow	◎
(iii) Joint	Discontinuous ground severed by joints or fissures, often found in hard rock		Properties of bedrock	△
			Degree and orientation of fissuring /cracking	◎
			Water inflow	○
(iv) Highly weathered rock or fracture zone	Locally deteriorated (e.g., faults, fracture zones) ground with properties that differ from those of the surrounding ground, such as weathered granite		Properties of bedrock	◎
			Degree and orientation of fissuring /cracking	△
			Water inflow	◎

Note: ◎: very important ○: important △: not very important

2) Monitoring results

The monitoring results should be evaluated to decide the following:

(i) Evaluation of the rock properties

Evaluation of the rock properties is made through direct measurement of the physical properties and state of the ground. Primary items are seismic surveys inside the tunnel, rock laboratory tests, and tunnel deformation monitoring.

(ii) Evaluation of ground deformation

Primary items include convergence, crown settlement, tunnel deformation, surface settlement, etc. The stability of the tunnel and the validity of the tunnel support are evaluated by monitoring of displacement behaviour of the ground. The techniques to utilize displacement monitoring results to evaluate/predict ground behaviour or the stability of the tunnel are classified

into the critical strain method and the method based on predicted final displacement.

In the critical strain method, ground strain is calculated from the monitoring results of ground displacement caused by tunnel excavation, and the value obtained is compared with the critical strain of the ground, resulting in the evaluation of ground behaviour. Figure 5-9 shows an example of a criteria proposal for controlling crown settlement by comparing the crown settlement monitoring results from the number of tunnels constructed in the past with the critical strain and integrating the comparison results.

The approach based on an expected final displacement involves monitoring of convergence occurring during one day immediately after the completion of the excavation, based on which the initial maximum displacement per day and the displacement rate (in millimetres per day) are used to predict at an initial stage of excavation the final displacement volume and the convergence timing so that necessary measures may be taken.

(mm)

	Class A	Class B	Class C
Precaution Level	3 – 10	5 – 15	10 -40
Warning Level	10 – 30	15 - 40	40 - 110
Critical Level	30 -	40 -	110 -

(tunnel diameter = 10m)

Class A Rock: σ_c > 100 MPa
Class B Rock: σ_c = 5 – 100 MPa
Class C Rock: σc = 0.5 – 5 MPa
σ_c: unconfined compression strength

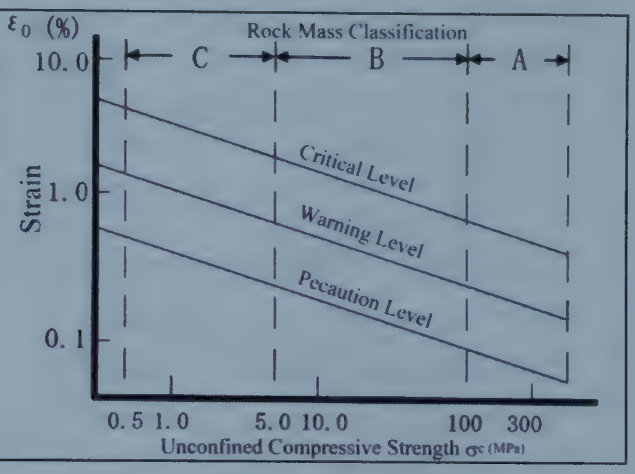


Figure 5-9: Example of Control level for Crown Settlement (Tunnel Diameter: 10 m)

(iii) Evaluation of the effectiveness of the support
The effectiveness of the function of each support system shall be evaluated in order to modify the tunnel support system in accordance with the rock condition. Stress (axial force) measurement data on respective support members are analysed to obtain the load sharing ratios among the members involved, and then the need for increase/decrease of respective support members is determined.

(iv) Determination of when to place the lining
The lining shall be placed after confirming through monitoring that the rock displacement has converged. Typically, displacement continues at a rate of about 1-3 mm per month (about 0.2 - 1 mm/week) for two weeks.

(3) Feedback to Design, Construction, and Maintenance

The monitoring results shall be evaluated comprehensively in accordance with the control level so they can be promptly incorporated into the design and construction. They shall be promptly fed back to the design and construction and records shall be kept and maintained in a well-organized manner for use as reference materials for future maintenance and operation.

1) Feedback to Design and Construction
The following procedures should be taken in order to make use of the monitoring results in the design and construction process.
   i) Establishment of control level
   ii) Monitoring as work progress
   iii) Estimation of final displacements from the monitoring results
   iv) Comparison of monitoring results and estimated values

v) Evaluation of the original design
vi) Review of control level

2) Feedback to maintenance
Monitoring results are helpful and useful for identifying the mechanism and causes of deformation in case that repairing works of support systems or additional reinforcement works are required. An example of a summary table commonly used for tunnelling work records is shown in Figure 5-10.

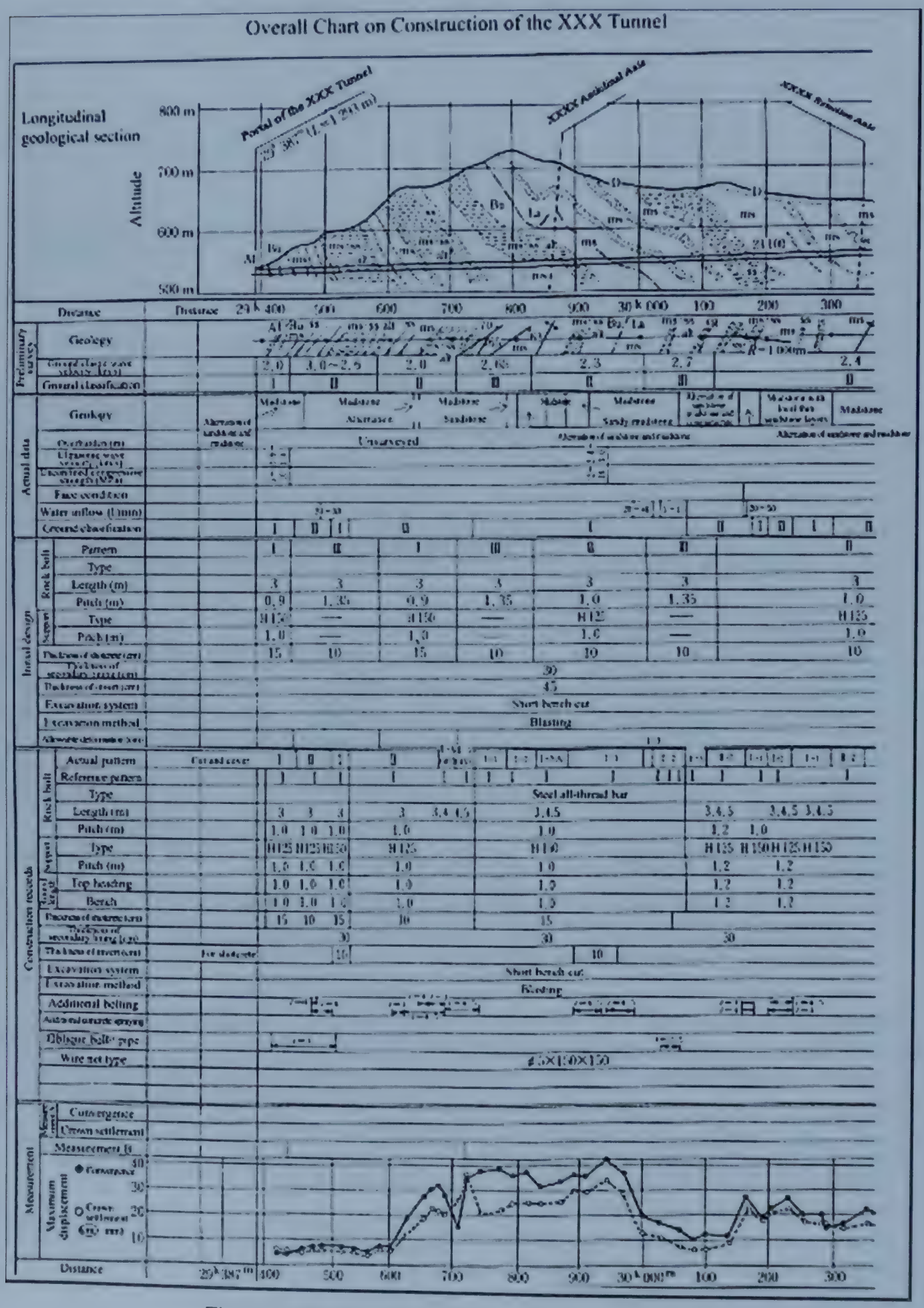


Figure 5-10: Example of Tunnelling Work Record

5.2.4 Instrumentation

Instruments shall provide functions and suitable for the purpose of monitoring, and shall be easy to install, handle and maintain.

(1) Ground Movements - Vertical & Lateral Deformations

The primary purpose for monitoring ground movements is to detect them while they are still small and to modify construction procedures before the movements grow large enough to constitute a real problem by affecting either the advancing excavation or some contiguous existing facility. For the advancing excavation, ground support has to be based on conditions encountered; monitoring either confirms the adequacy of the support or indicates whether more or different support may be required. Existing facilities may be at the ground surface - roads, railroads, buildings, and the like - or they may be below ground in the form of utilities or other transportation tunnels such as subways. The first line of defence against potentially damaging movements is to detect them at depth in the ground immediately surrounding the advancing tunnel and take mitigative action before those movements can "percolate" upward toward the surface. This kind of monitoring can provide an indication of whether ground treatment such as grouting is effectively limiting movements that might otherwise result in troublesome settlements. The ground can, of course, move upward as well as downward, in the form of heave from unloading that can destabilize the invert of the tunnel under construction, and as a side effect lead to lateral, possibly damaging deformations as the ground moves toward the excavation to take up the slack. In addition to helping control the ground, the data developed can be used (and this may be said of all monitoring discussed in succeeding paragraphs) to verify design assumptions and to evaluate claims by construction contractors and third-party abutters. Several types of instrumentation are used to monitor ground movement:

- Deep Benchmarks
- Survey Points
- Borros Points
- Probe Extensometers
- Fixed Borehole Extensometers, either measured from the surface or during the advance of the tunnel
- Telltales or Roof Monitors
- Heave Gages
- Conventional Inclinometers
- In-place Inclinometers
- Convergence Gages

1) Deep Benchmarks

Deep Benchmarks (Figure 5-11) are steel pipes/casings drilled into stable strata - preferably sound bedrock - outside the advancing tunnel's zone of influence. They are used when existing benchmarks are not available and it is important to know actual elevation changes of other instruments meant to detect movements. If installed close to the construction, deep benchmarks need to be carried below the invert. They must be absolutely stable in spite of any ground movements that are occurring because it is the surface level collars of these devices that become the unmoving points from which locations and elevations of other instruments can be determined by surveying. A major complication in the installation of benchmarks can be the difficulty of installing them in a location and/or to a depth that absolutely guarantees no movement as tunnelling proceeds. In this regard the lowering of groundwater in a soft ground environment can contribute to ground settlements well outside the immediate projected footprint of the advancing tunnel, so the instrument has to be well placed to guard against this eventuality. In cases of very large projects or overlapping projects that cause the water table to be drawn down across a large area, benchmarks have been known to settle even when

founded in bedrock because some rock types can be dependent to a degree on pore water pressure for their ability to carry the load.

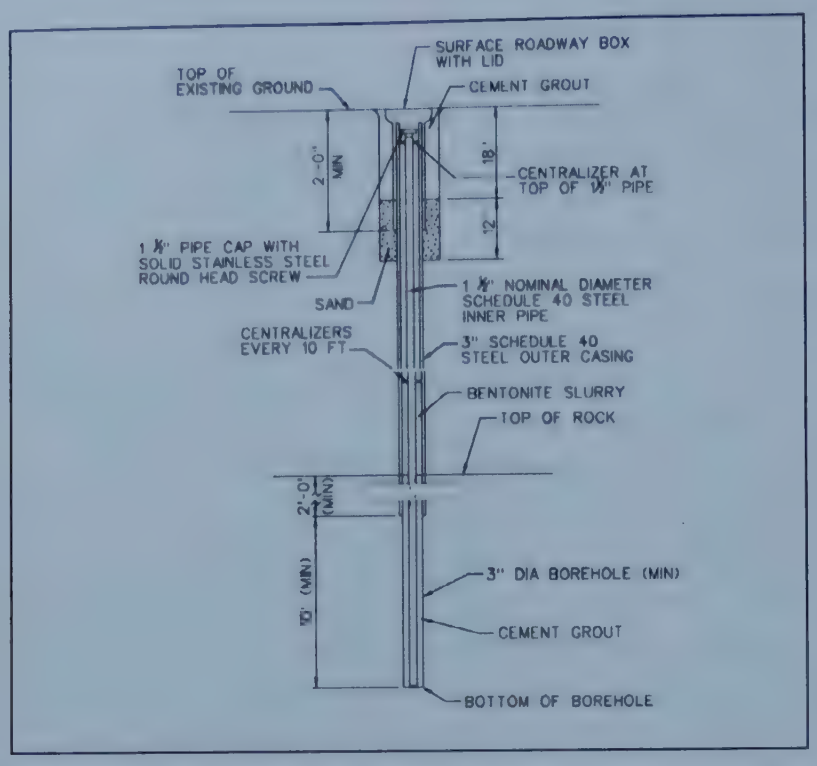


Figure 5-11: Deep Benchmark[1]

2) Survey Points

Survey Points are used to detect ground movements at the surface or shallow depth below the surface. They may be as simple as wooden stakes driven into the ground and their elevations surveyed through backsighting to a deep benchmark (Figure 5-12). Penetration needs to be at least a foot or so to guard against dislodgment, and the tops should not extend high enough to interfere with mowing machines if they are in a grassy area that requires routine maintenance. A survey point may also be somewhat more sophisticated and take the form of a steel rod with a rounded reference head driven into the ground for better avoidance of possible dislodgment and surface effects such as frost heave (Figure 5-13). This type of point needs to be protected at the surface by a small utility-type road box with a secured cover so there is no disturbance to the rounded head. A rounded head is considered best because a surveyor can then always find the high point that has been surveyed in the past for good continuity in the readings. Because there is no hard connection between the rod and the road box - the one sort of "floats" inside the other - the survey point is also protected from being pushed down in case of the passage of a heavy vehicle. The major concern with any type of survey point is the need to keep it out of the way of other users of the area and also protect against damage that may require replacement and lead to loss of continuity between the latest reading and the string of readings taken in the past.

[1] U. S. Department of Transportation Federal Highway Administration (2009); Technical Manual for Design and Construction of Road Tunnels - Civil Elements

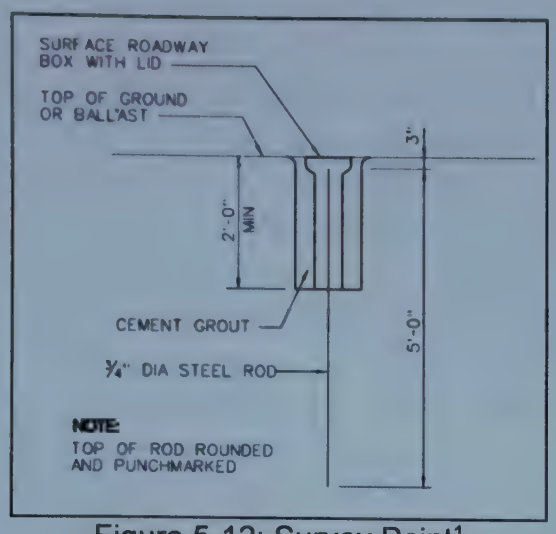


Figure 5-12: Survey Point[1]

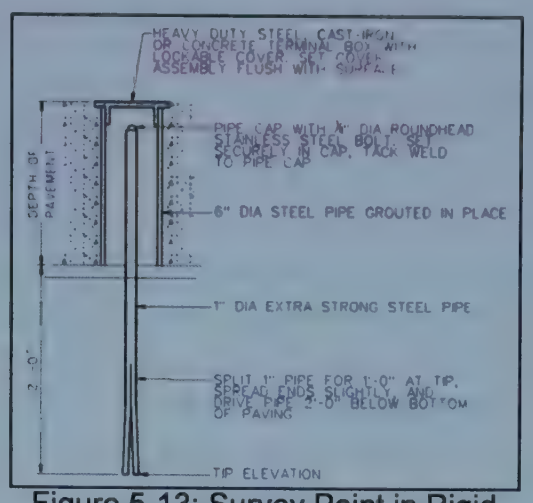

HEAVY DUTY STEEL CAST IRON
OR CONCRETE TERMINAL BOX WITH
LOCKABLE COVER SET COVER
ASSEMBLY FLUSH WITH SURFACE
PIPE CAP WITH ¾" DIA ROUNDHEAD
STAINLESS STEEL BOLT SET
SECURELY IN CAP, TACK WELD
TO PIPE CAP
DEPTH OF
PAVEMENT
6" DIA STEEL PIPE GROUTED IN PLACE
1" DIA EXTRA STRONG STEEL PIPE
2'-0"
SPLIT 1" PIPE FOR 1'-0" AT TIP,
SPREAD ENDS SLIGHTLY AND
DRIVE PIPE 2'-0" BELOW BOTTOM
OF PAVING
TIP ELEVATION

Figure 5-13: Survey Point in Rigid Pavement Surface[1]

3) Borros Points

A Borros Point is basically an anchor at the lower end of a driven pipe (Figure 5-14). The anchor consists of three steel prongs housed within a short length of steel pipe with points emerging from slots in a conical drive point. Installation is achieved by advancing a borehole in soft ground to about one meter above the planned anchor depth and the anchor inserted by attaching extension lengths of the riser pipe and outer pipe. When the point reaches the bottom of the hole, it is driven deeper by driving on the top of the outer pipe. The prongs are then ejected by driving on the riser while the prongs are released and the outer pipe bumped back a short distance to achieve a positive anchorage. Such installations are useful for determining the amount of settlement at one precise depth with more certainty than the simply driven steel rod described above, and they are relatively simple and economical. The amount of anchor movement is determined by surveying or otherwise measuring the movement of the inner riser pipe at the ground surface. One disadvantage with such movement detection (and this can be said of most instruments whose data depends on movements measured in a surface mounted reference head) is that, if settlement is great enough to have affected the surface at reading time, then the whole instrument may be moving downward by a certain amount while the anchor is moving downward by a greater amount. Absolute anchor movement may then be difficult to judge unless ground elevation surveys are undertaken at that time and the changes added to the apparent anchor movement.

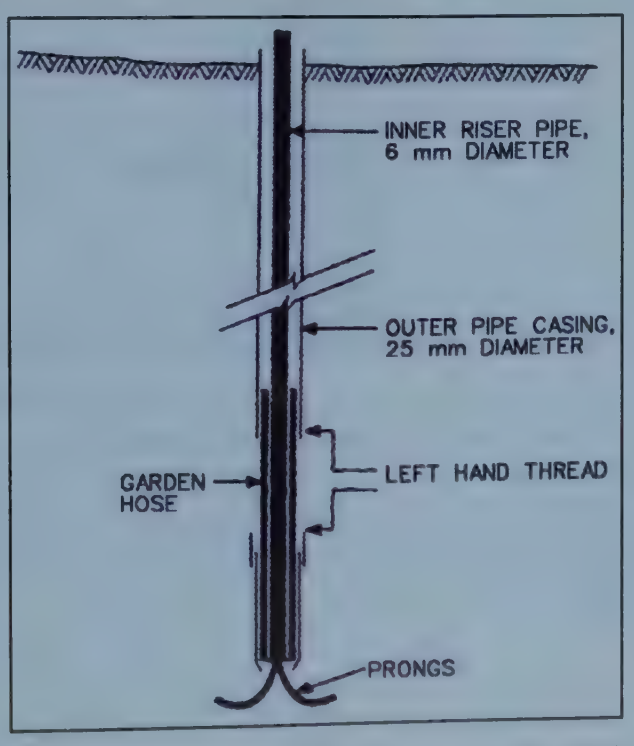


Figure 5-14: Schematic of Borros Point[1]

4) Probe Extensometers

Probe Extensometers are used to measure the change in distance between two or more points within a drilled hole in soft ground, through the use of a portable probe containing an electrical transducer. As shown in Figure 5-15, the probe, which contains a reed switch, is inserted into a casing in the drill hole in which the reference points, each of which contains an array of bar magnets, have been fixed in a way to surround the casing on the outside. In the most common type of installation, the reference points are held in place by spring-loaded anchors - leaf springs - that "bite" into the ground. The points are free to move with the ground because the outer support casing will have been removed and replaced by grout. The probe detects the depth of the reference points for an indication of whether the soil at those depths is settling due to disturbance from construction. A probe extensometer can thus measure the settlements at a much larger number of depths than can a Borros Point. Probe extensometers are generally drilled to a depth below any potential zone of influence near a cut-and-cover or mined tunnel. The bottom reference point then becomes the unmoving reference from which the movements of the shallower points are judged. In a typical situation near a mined tunnel, it is likely that the lowest moving point will exhibit the most settlement, and that settlements will prove to be less as the probe moves up the casing to where the settlement trough is widening. One problem with probe extensometers is that collection of data can be operator sensitive as the instrument reader strains to detect the exact location of the probe at each reference point depth by listening for the electronic "beep" to ensure readings at precisely the same spot time after time. Another concern may be the time required for monitoring, especially if a large number of reference points have been installed because the probe does have to be lowered to the bottom of the casing and then readings collected as it is slowly winched back to the surface.

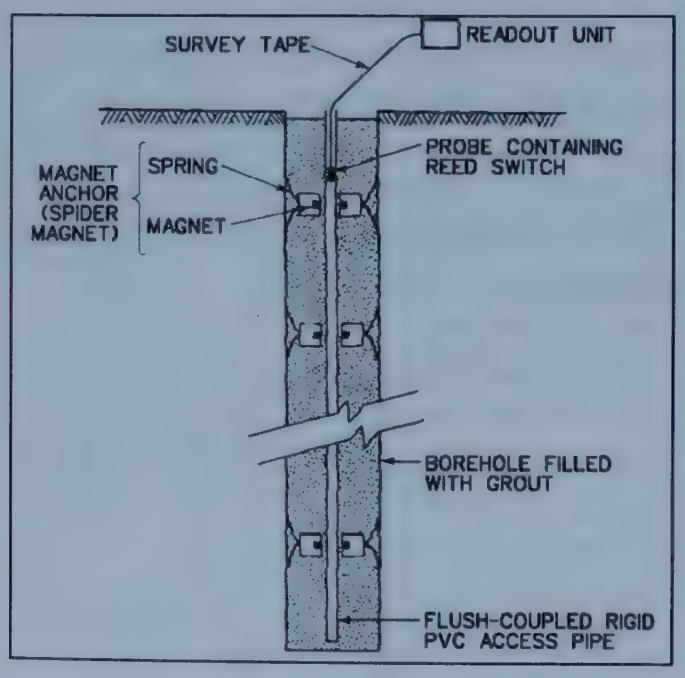


Figure 5-15: Schematic of Probe Extensometer
with Magnet/Reed Switch Transducer, Installed in a Borehole[1]

5) Fixed Borehole Extensometers

Installed from Ground Surface Fixed Borehole Extensometers installed from the ground surface may be used in soft ground or rock and may be Single Position (SPBX) for settlement measurements at one specific elevation or Multiple Position (MPBX) for measurements at several elevations. Figure 5-16 illustrates a schematic of an MPBX. The anchors of a borehole extensometer are grouted into the ground, commonly at various distances above the crown of an advancing tunnel, and connected to surface-mounted reference heads by small diameter rods of steel or fiberglass. By detecting movement of

the tops of the rods at the surface, one can tell how much each anchor - and hence its increment of soil or rock - is moving in response to excavation and so take steps to mitigate developing problems. Manual readings can be taken in a matter of minutes, assuming there is no problem with access to the instrument collar. However, automatic readings with an electrical transducer and datalogger are relatively inexpensive and can provide real-time data that feeds directly and quickly into a computer for fast analysis and databasing. Although extensometers oriented vertically over mined tunnel crowns are the most common installations, two others may prove useful in particular situations: (a) instruments angled in toward tunnel crowns or haunches from sidewalks where vertical installations are precluded by heavily travelled roads; (b) instruments installed along the sidewalls of mined tunnels or cut-and-cover excavations where a knowledge of the vertical component of overall ground movement may be advantageous. A common problem with manually read instruments is the one of operator sensitivity, and if more than one reader is employed, they need to practice together to make certain they can monitor with good consistency. Remote monitoring leads to the concern that data collectors and analysers may, without themselves personally having an eye on the construction operation, be unaware of the type and schedule of activities that are affecting the data. Hence it may be necessary to make arrangements for construction progress reports to be delivered on a tighter schedule than otherwise might be necessary.

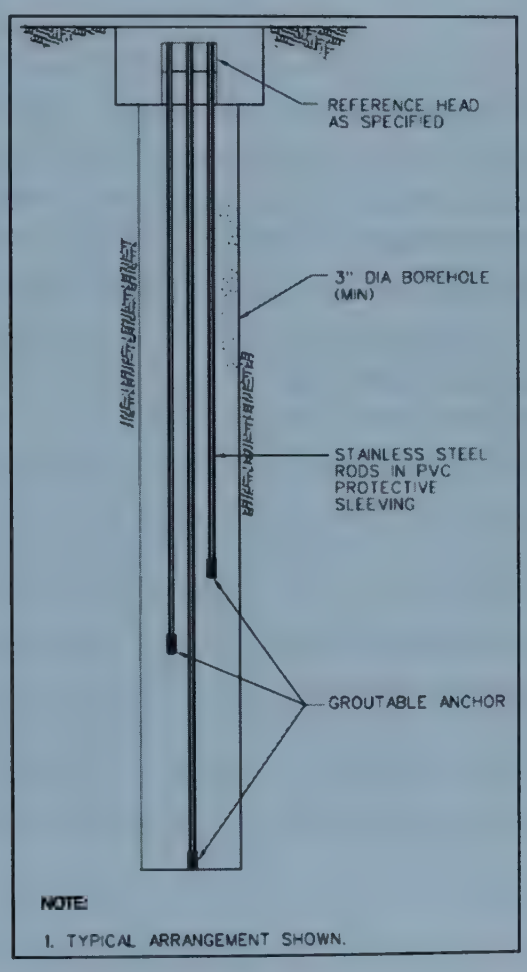


Figure 5-16: Multiple Position Borehole Extensometer Installed from Ground Surface[1]

6) Fixed Borehole Extensometers Installed from Advancing Excavations

Fixed Borehole Extensometers installed from advancing excavations are a fairly obvious need if sidewall movements are required for a cut-and-cover excavation. Such horizontal installations are common and the drilling/installing operation has to mesh with the construction so that the larger operation is not overly impacted by what may appear to be a peripheral activity. (Note: "Horizontal" installations are seldom truly horizontal because angling downward by 10 or 15 degrees makes it much easier to manage the grouting of the anchors.) The installation of extensometers oriented from the vertical to the horizontal - including all angles in between - from inside advancing mined rock tunnels may be mandated by the lack of access from the ground surface (Figure 5-17). In this

position, they can provide data on incipient fallouts or more subtle rock movements toward the opening. If installed where a small tunnel is to be enlarged to greater size at a later time, the instrument heads can be recessed beyond the initial excavation outline and saved for use in monitoring the larger excavation. In this way, they provide an almost complete history of rock movements from the earliest to the latest point in time. Another way to use these instruments is to install them from a first driven tunnel toward the location of the following twin tunnel. Readings then indicate whether the pillar between the two tunnels is loosening so that steps can be taken to mitigate the problem.

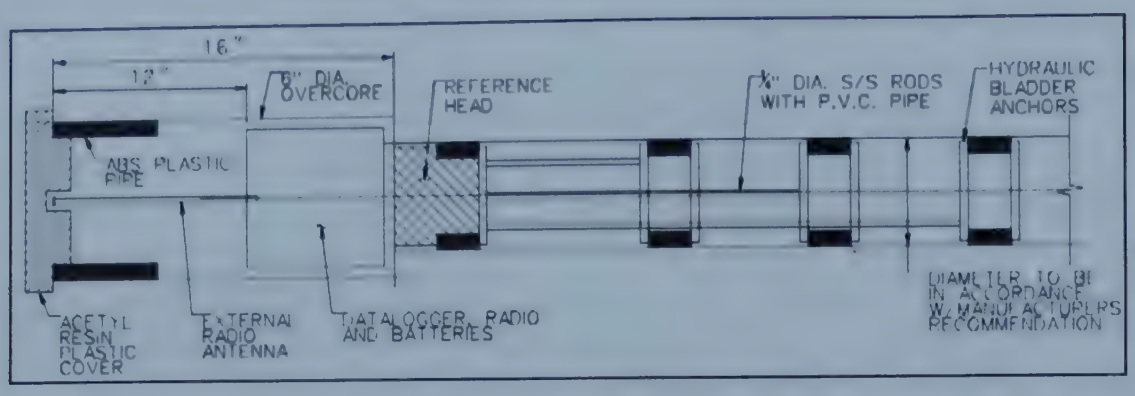


Figure 5-17: Horizontal Borehole Extensometer Installed from Advancing Excavation[1]

Complications for these in-tunnel instruments are more numerous than for those installed from somewhere outside the excavation. As noted, the installation has to mesh with the construction operation, a particularly tricky proposition in the confines of a small-mined tunnel, where constructor complaints of interference are extremely common. Even the collection of data, if it is performed manually, may be obtrusive, especially if the cessation of tunnelling, use of ladders, or help from constructor personnel are involved. Remote monitoring is also possible, but then there is electrical wiring to be run and the need to find a place for the datalogger(s) to be out of the way. By whatever method the in-tunnel instruments are monitored, the reference heads need to be protected, often by countersinking them in the tunnel wall and perhaps through the installation of protective covers. This is especially true where there is going to be more blasting in the vicinity, but also true even where blasting is not involved. Miners tend to have little reverence for objects whose importance is not obvious to them, so vandalism and theft of instrument accoutrements have to be guarded against. Finally, there is the fact that an in-tunnel instrument is almost always installed after the tunnelled ground has started to relax, so the initial readings are seldom true zero points from which to compute follow-on movements. The instrumentation specialist's only recourse is to continually press the constructor for access to install instruments at the earliest possible opportunity.

7) Telltales or Roof Monitors

Telltales or Roof Monitors (Figure 5-18) are other devices that can be installed from inside an advancing rock tunnel. They are designed to be installed with anchors in stable rock beyond the tips of rock bolts in tunnel roofs to provide fast feedback on stability. The immediate safety of the miners/tunnelers is the primary reason for the instrument's use. The first ones were steel rods with a single anchor and visual movement indicators in the tunnel roof that could be seen by miners as they worked. Simple and installable by rock bolting crews, they proved vulnerable to shearing due to the movement of rock blocks and were eventually replaced by more flexible steel wires that are less prone to failure. Modern versions have as many as three anchors and can be wired for remote reading by a trained person watching the data on a laptop computer. Roof monitors are widely used around much of the world, where they deserve to join the ranks of commonly used instruments. They are now used in civil as well as mine construction and also in rock other than flat-lying sedimentaries commonly associated with coal seams.

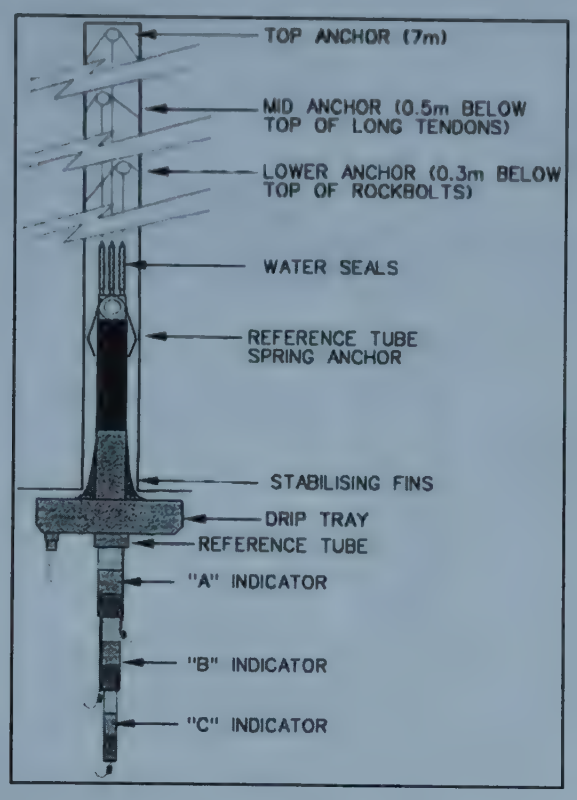


Figure 5-18: Triple Height Telltale or Roof Monitor[1]

8) Heave Gages

Heave Gages are most commonly used when excavating for open cut or cut-and-cover in soft clay where there is potential for the bottom to fail by heaving as overburden load is removed. There are several instruments with which heave can be detected and measured, but almost all either suffer from lack of accuracy or are prone to damage or malfunction. Interestingly, the magnet/reed switch gage packaged as for a probe extensometer is probably the best alternative (Figure 5-19). In this type of installation, the user measures increasing rather than decreasing distances between spider magnets and a fixed bottom anchor. With care taken to make certain the bottom anchor is well below any expected zone of movement, the installation is made inside the cofferdam prior to the start of excavation. After initial readings are taken the access pipe is sealed 1.5 to 3 meters below the ground surface through the use of an expanding plug set with an insertion tool, and the pipe is cut with an internal cutting tool just above the plug. A good fix is made on the plan location of the instrument and, just before the excavation reaches the plug, the pipe is located, a reading made, and the pipe again sealed and cut. The procedure is repeated until the excavation is complete. The concern with such installations - a concern not overcome with alternative installation types - is that any large excavation is made by means of heavy equipment, and operators are not prone to watching and caring for things as small as a heave gage pipe. It is common for the gages to be damaged beyond use, and their protection can be assured only through some forceful work control and sometimes the levying of penalties for instruments damaged as a result of contractor carelessness.

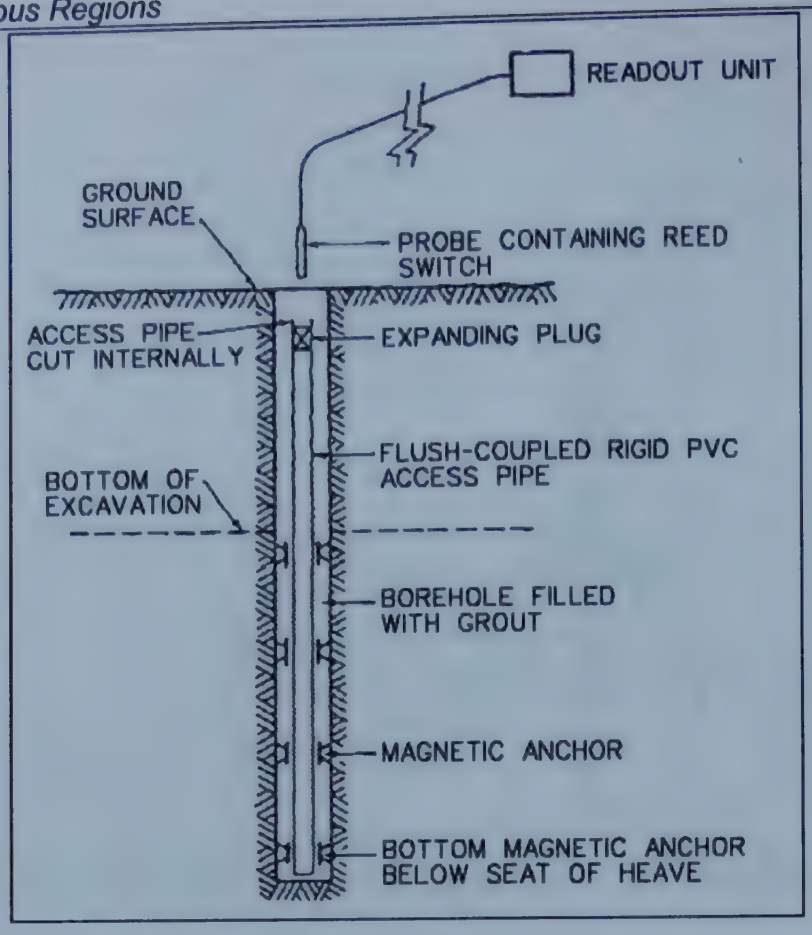


Figure 5-19: Heave Cage[1]

9) Conventional Inclinometers

As shown in Figure 5-20, conventional Inclinometers are aluminum or plastic casings drilled vertically to below the level of construction into a stable stratum and used to determine whether the surrounding ground, either rock or unconsolidated material, is moving laterally toward the excavation. Each casing has tracking grooves to guide the sensing probe for orientation both parallel to and at right angles to the axis of the excavation. The probe, which contains tilt sensors, is lowered on a graduated cable to the bottom of the hole and winched upward, with stops at 2-foot intervals for collection of inclination data by means of a readout unit at the ground surface. An iterative process of tilt calculations from the unmoving bottom of the casing permits the plotting of a profile that fixes each measured increment of casing in space in relation to the excavation. An initial set of inclination readings is taken before excavation begins and each set of readings thereafter during construction provides data on how the ground is moving when the user plots the newer movement curves against the initial pre-construction curves. The inclinometers are normally situated a close distance from the excavation periphery of open cut or cut-and-cover excavations, but may also be installed just outside a mined tunnel where lateral movement data may be combined with vertical movement data from the extensometers discussed above. The term "conventional inclinometer" is used herein to distinguish the manually read instrument from the "in-place" instruments described below. The major concern with a conventional inclinometer is the time consumed in the monitoring process. Readings are performed twice in each monitoring visit, once with the probe inserted in the "A" direction tracking grooves, then again with the probe in the "B" direction. A "check sum" procedure is carried out by examining the sum of the two readings at the same depth, 180 degrees apart, in order to remove any long-term drift of the transducers from the calculations. It commonly requires 45 or so minutes for a reader to collect data from a 100-foot deep instrument, and that is assuming no indication of excessive movements, which, if discovered, may require another set of readings for confirmation that the movements are real and not due to a reading error or instrument malfunction.

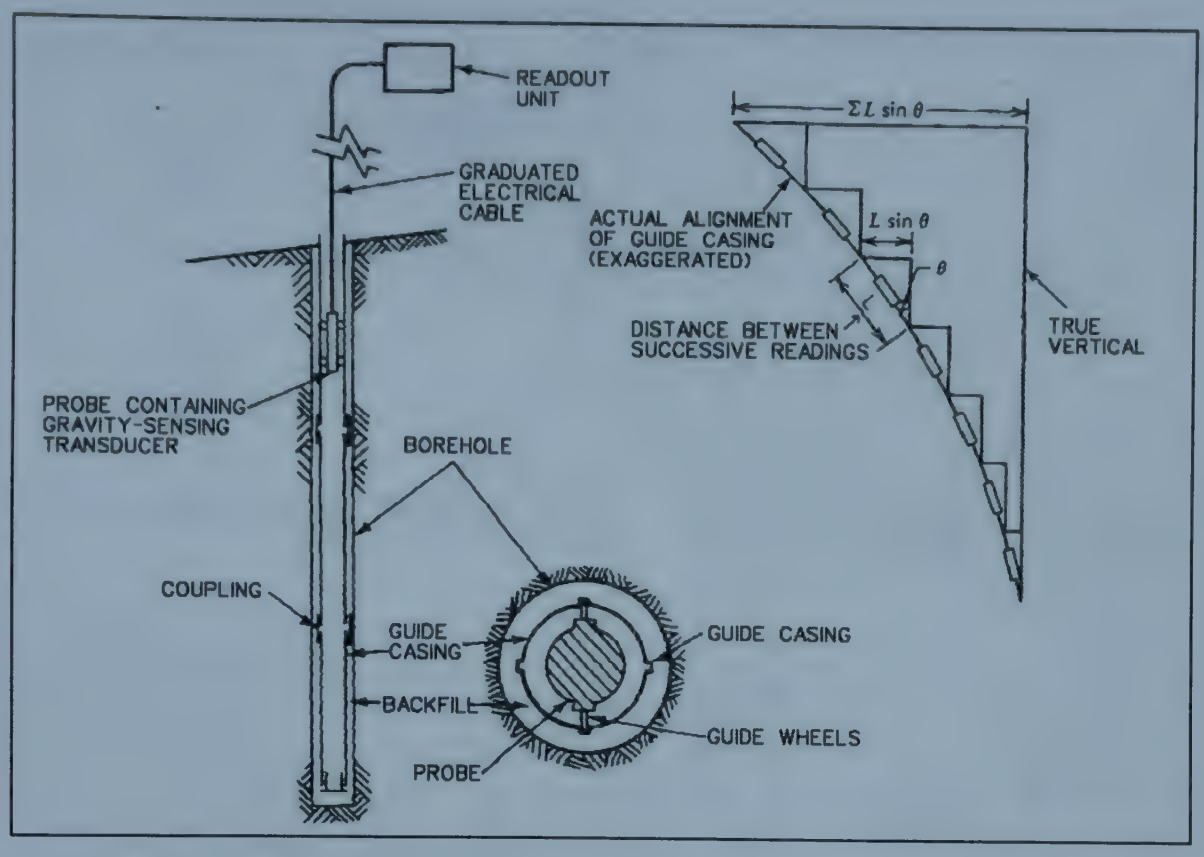


Figure 5-20: Principal of Conventional Inclinometer Operation[1]

10) In-Place Inclinometers

In-Place Inclinometers are typically used for monitoring subsurface deformations around excavations when rapid monitoring is required or when instrumented locations are difficult to access for continued manual readings. The sensors are computer-driven, gravity-sensing transducers joined in a string by articulated rods, and they can be installed equidistantly in the casing or concentrated in zones of expected movement (Figure 5-21). With the in-place instrument, as many as ten or twelve sensors are mounted in the casing and left semi-permanently in place. A larger number of sensors would be difficult to install in a standard-size drill hole because each sensor has its own set of signal wires that take up space, and a very large number of sensors could result in the need for an uneconomically large diameter drill hole. Signals are fed to a datalogger at the surface and can be collected as often as required, or even fed by telephone line to the database computer for something close to real-time monitoring. Compared with conventional instruments, the in-place inclinometer hardware is expensive and complex. This can sometimes be compensated to a degree by removing sensors from a bypassed instrument and installing them in a new location as the excavation progresses. A not-so-easily-overcome disadvantage of the in-place instrument lies in the fact that, if there is any long-term drift in any of the sensors, it cannot be overcome through the check sums procedure described above. It is also true that the somewhat limited number of sensors in a standard in-place installation leads to a less smooth plot of movements compared with what can be achieved with the conventional inclinometer.

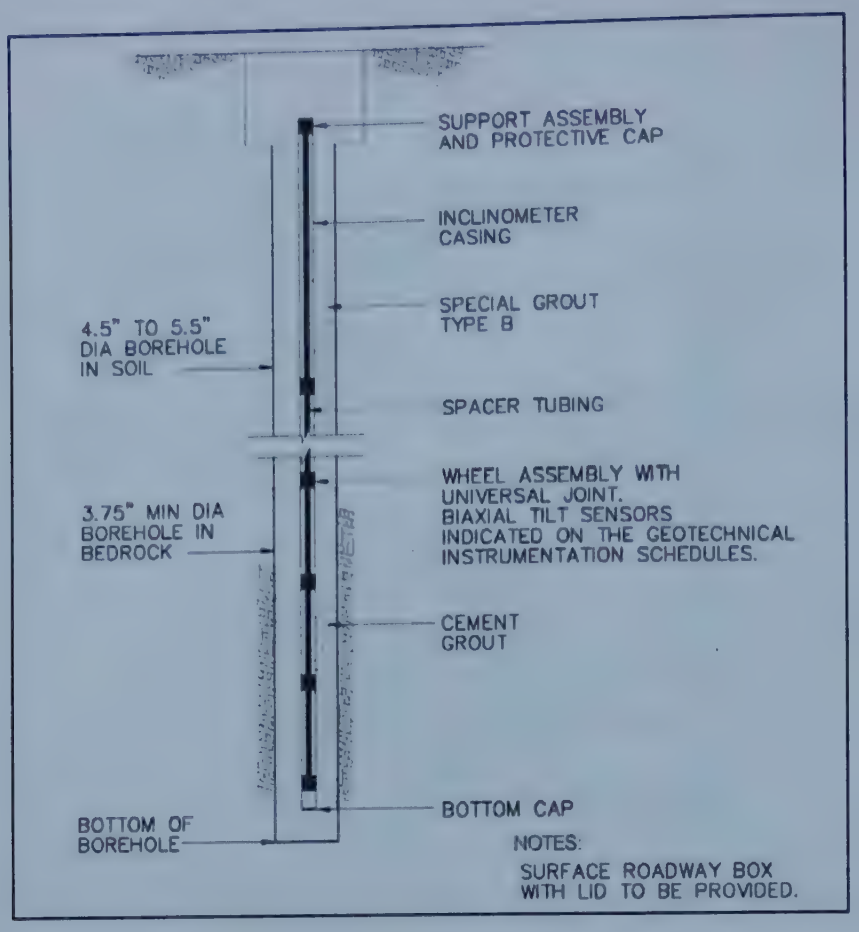


Figure 5-21: In-Place Inclinometer[1]

11) Convergence Gages

Convergence Gages may be used for monitoring the closure of the ground across either open excavations or mined tunnels. In the first instance, they perform a function similar to an inclinometer, although with many fewer data points to give a full picture of movements. In the second function, they detect the load redistribution during and after excavation and the extent to which resulting structure/ground interaction affects the tunnel shape and the lining. Until now the typical gage has been a Tape Extensometer, which includes steel tape with holes punched at 50 mm intervals (Figure 5-22). Anchors that define monitoring points consist of eyelets on the ends of grouted rebar sections that extend into the ground for a foot or so (Figure 5-23). The tension in the tape is controlled by a compression spring, and standardization of tension is achieved by rotating the collar until scribed lines are in alignment. After the attachment of the extensometer to the anchors and standardization of the tension, readings of distances are made by adding the dial indicator reading to the tape reading. In a typically mined tunnel, the pattern of anchors includes one in each sidewall at springline level and one as close as possible to the centre of the tunnel crown. Three readings are taken in a tent-shaped pattern and the results indicate whether the tunnel support is behaving in a predictable way. For very large tunnels, the patterns may be more like trapezoids or overlapping triangles, which requires the installation of additional anchors. Such readings are only relative readings, and if absolute elevation changes are needed, this is usually accomplished by surveying the anchor that is in the crown. (Installation directly in the high point of the crown is seldom possible because of the presence of the ventilation and other lines.)

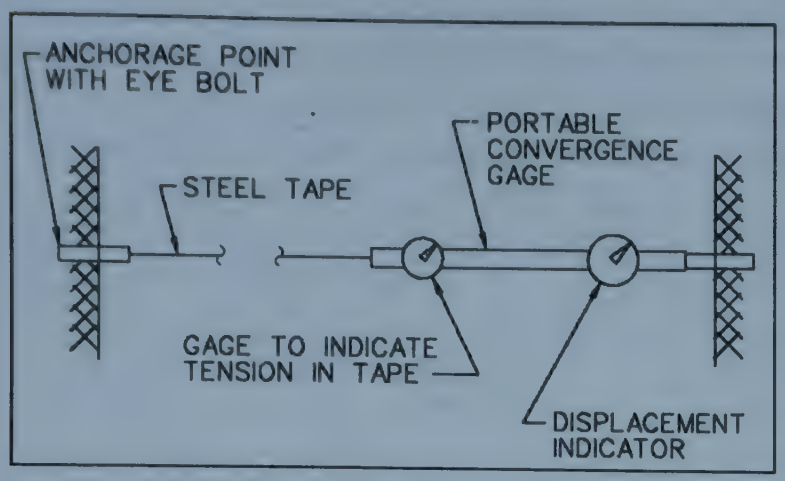


Figure 5-22: Tape Extensometer Typical Detail[1]

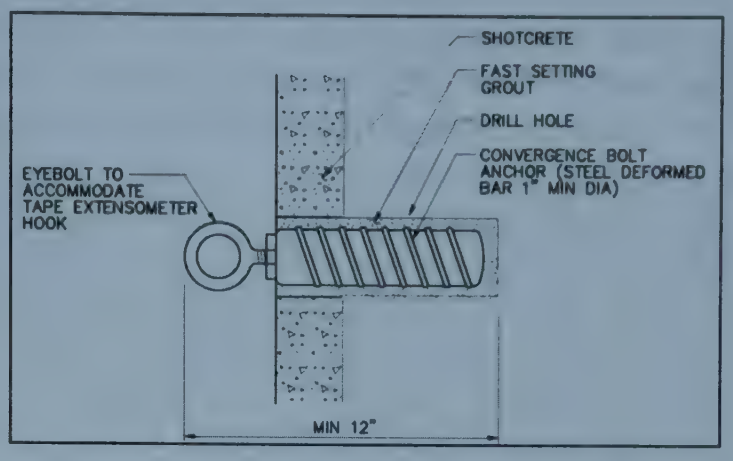


Figure 5-23: Typical Convergence Bolt Installation Arrangement[1]

(2) Monitoring of Existing Structures

If the different parts of a structure should move uniformly by even large amounts, the damage could be minimal, maybe non-existent, except perhaps for penetrating utilities such as water pipes that might not be able to accommodate themselves to such movements. However, most structures affected by construction react by exhibiting more movement of the parts closest to the excavation than of the parts that are further away. This differential movement is the principal cause of construction-related damages because the affected structure may be subjected to forces it was not designed for. A building, for example, whose footings are settling on one side while the other side settles less or not at all will suffer tilting of some walls, and the racking that ensues may cause cracking or spalling of some architectural features, freezing of doors and windows, or, in the worst case, failure of one or more of the structural members. A bridge whose footings are subjected to differential movements may undergo extensions that literally tear it apart. In general, the detection of settlements is the first line of defence in the protection of existing facilities, whether they be surface (roadways, buildings, bridges) or subsurface (utilities, transit tunnels, other highway tunnels). The detection of tilting can also be useful and has become more common as the development of monitoring devices has proceeded in the direction of increased automation. The simplest kind of monitoring involves the detection and tracking of joint separations and crack propagation in structural concrete or architectural finishes. The ideal is to detect and mitigate some or all of these movements before they have become severe enough to cause serious damage or perhaps constitute a hazard.

As with ground movement instrumentation, there are a number of choices of instrumentation:

- Deformation Monitoring Points
- Structural Monitoring Points
- Robotic Total Stations
- Tiltmeters
- Utility Monitoring Points
- Horizontal Inclinometers
- Liquid Level Gages
- Tilt Sensors on Beams
- Crack Gages

1) Deformation Monitoring Points

Deformation monitoring points on roads, streets, or sidewalks can be as simple as paint marks that get surveyed on a routine basis. However, paint has the disadvantage that it can be visually obtrusive, may wear off with time, and may not display a single spot that surveyors can return to time after time for good data continuity. A better alternative is a small bolt-like device set in an expansion sleeve that can be installed in a small hole

drilled in concrete or asphalt. The point should have a slightly protruding rounded head with a consistent high point that is always findable by a surveyor as he or she searches for the same unchanging spot on which to set the stadia rod. It is important that the point does not protrude too much because it might then become a tripping hazard or be vulnerable to damage from equipment such as snow plows. Although they are inexpensive to purchase and install, the ultimate cost of deformation monitoring points can grow to become relatively high if data collection becomes intensive because it depends upon the mobilization of survey crews. Also, such monitoring is not always foolproof because surveyors are not necessarily attuned to the need for that high degree of accuracy that is sought by instrumentation specialists. It is very common for data thus generated to exhibit a fair amount of "flutter," i.e., apparent up-down movements that are not real but are only the result of inconsistencies in the survey process. Such inconsistencies may result from the too-often changing of personnel in survey crews, changes that happen commonly due to the nature of the business. Luckily, extreme accuracy is not required in much of this paved surface monitoring, so if the surveyors can reliably detect changes of 5 cm or so, that is often good enough.

2) Structural Monitoring Points

Structural Monitoring Points are survey points that are placed directly on the structures of concern, most often being installed on a vertical wall of a building or a structural element of a bridge. Except for buildings, most structures can accommodate the monitoring point likely to do the best job and the "points" may take several forms. The simplest is a tiny scratch mark that can be easily found on each monitoring visit by a survey crew. A similar point is a stick-on decal target, which is a bit more obtrusive, but easily removable once it is no longer needed. A problem with such surface treatments is that for buildings particularly, the monitoring point may be only on a facade that moves independently of the underlying structural elements whose movements it is important to detect. This may be overcome by the installation of a bolt-like device that penetrates to the underlying structure for a truer indication of the movements taking place. The choice of monitoring points will often depend on the wishes of owners or managers of buildings who may object to the visual obtrusiveness or potential for damage from whatever may be installed. Possible damage can extend to the post-construction period when the monitoring point may have to be removed and patched, something that is often insisted on by the party who permitted its installation. Thus, it may be necessary to repair the scars left by the removal, which may entail the use of solvents, infilling, sparkling, polishing, painting, or replacement for satisfactory restoration.

A large consideration in the use of structural monitoring points is the need to depend upon surveyors for the collection of data. Compared with roads and sidewalks, most structures have tight specifications on permissible movements (a lower mitigation-triggering level of 5 mm being not unusual), so surveying generally needs to be of a somewhat higher order, not necessarily as stringent as Class I, but at least done with additional care. One way to achieve this is to hold briefings in which the importance of great accuracy is instilled in the surveyors who will do the work. Another (if it is possible in the economic climate of the day) is to write and enforce the survey contract so that each group of structures is always monitored by the same crew using exactly the same equipment. In this way, the "flutter" may be reduced so as to minimize the need for instrumentation interpreters to average the peaks and valleys in determining if settlements are real or only apparent.

3) Robotic Total Stations

Robotic Total Stations are used for obtaining almost real-time data on movements in three dimensions when it is not feasible to continually mobilize survey crews to collect data. The operation of a Total Station instrument (theodolite) is based on an electronic distance meter (EDM), which uses electromagnetic energy to determine distances and angles with

a small computer built directly into the instrument. Accuracy is generally much greater than that achievable with the use of classical optical surveying. Moreover, the equipment based on EDMs is capable of detecting target movements along all three possible plotting axes, the x, the y, and the z. Total stations used in geotechnical and structural monitoring are electro-optical and use either lasers or infrared light as the signal generator.

Robotic (also called automated motorized) total stations are configured to sit atop small electric motors and to rotate about their axes. As shown in Figure 5-24, they are mounted semi-permanently and, at predetermined intervals, automatically "wake up" to aim themselves at arrays of special glass target prisms (Figure 5-25) that can provide good return signals from a variety of angles. The target prisms, which are 5 to 8 cm in diameter, are installed on structures of concern, and the total station instruments are installed on other structures as much as 100 m away. It is best to have the total stations installed outside the expected zone of influence for absolute certainty of measuring target movements with accuracy. However, it is standard practice to install some of the prisms definitely outside the influence zone so that they become reference points from which the total station can determine its own position and calculate the positions of the other prisms that may be subject to movement. Clear lines of sight from the total station to target prisms are a requirement so that careful planning is required for proper placement. Data is recorded by means of the total station's own computer and may be fed to a centralized database computer by means of telephone lines or radio signals.

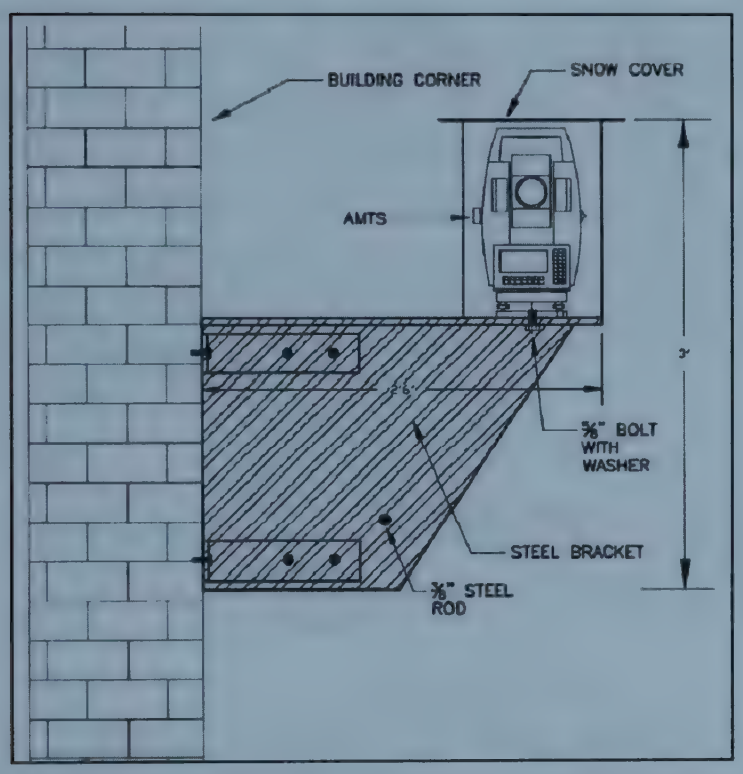


Figure 5-24: Robotic Total Station Instrument[1]

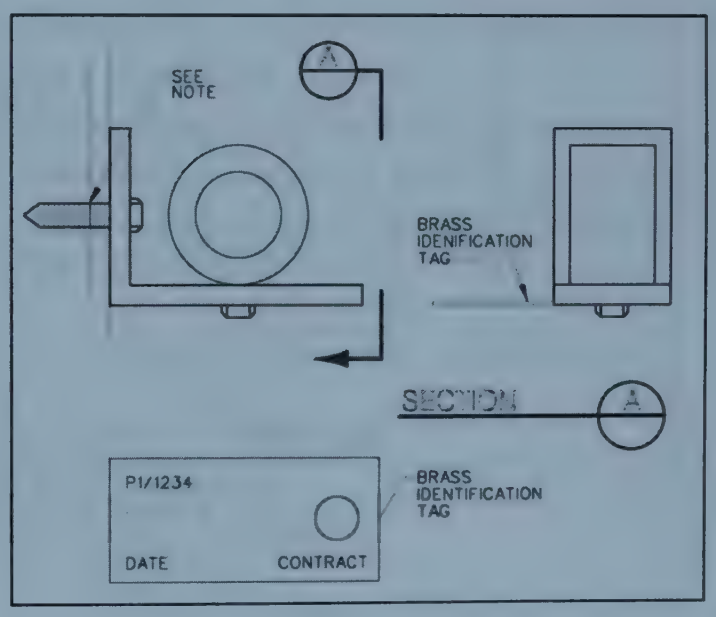


Figure 5-25: Target Prism for Robotic Total Station[1]

4) Tiltmeters

Tiltmeters are used to measure the change in inclination of structural members such as floors, walls, support columns, abutments, and the like, which may tilt when the ground beneath is being lost into an advancing excavation. Manual tiltmeters generally consist of reference points on plates attached to the surface of interest and monitored by means of a portable readout unit, the functioning of which is based on an accelerometer transducer. Because such an arrangement can be operator sensitive and reading is somewhat labour intensive, especially where continued access is not easy, it is becoming more common to collect data remotely by means of electrically powered tiltmeters whose sensing elements may consist of accelerometer or electrolytic level transducers placed in housings that can be attached to the element to be monitored. If only one direction of movement is expected, the chosen instrument may be uni-axial, but if there is a possibility of combinations of movement, the bi-axial instrument would need to be used. Figure 5-26

illustrates a biaxial tiltmeter. Because tiltmeters can inform users only about rotational components of movement, data must be combined with that from other instruments to determine levels of settlement that may be affecting the structure. The most difficult tiltmeter installations are those required for structural elements somewhere inside a building that is occupied. Even the manually read instrument, with a flat 15 to 20 cm diameter plate being the part attached, is somewhat visually obtrusive and may be objected to by a building manager. Remotely read tiltmeters are even more obtrusive because they need to be wired for electric power and connected to a powered datalogger that will probably need to have telephone connections if true real-time data is needed. There is some controversy within the monitoring community about the best installation height for these instruments, with some opting for lower floors and some for higher floors where absolute wall movement - though perhaps not *tilt* per se - will be greater. The argument is often laid to rest by a building manager who will permit such installations only in basement levels to better keep them out of the way.

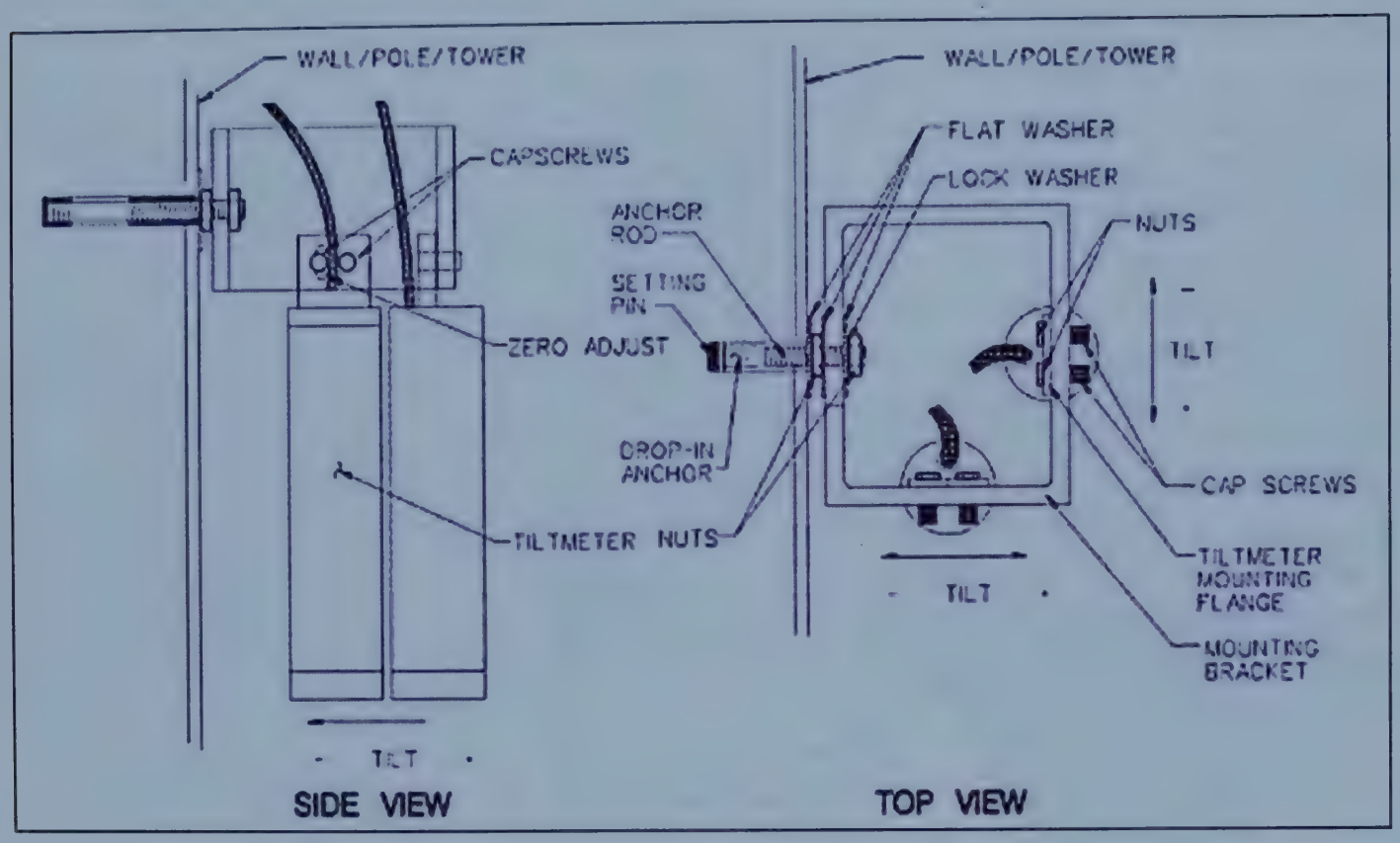


Figure 5-26: Biaxial Tiltmeter[1]

5) Utility Monitoring Points

Utility Monitoring Points are very simple instruments used to determine whether an existing utility such as a water line is settling in response to an excavation proceeding nearby or underneath. The device consists of a small pipe with a rounded survey point or arrangement for use of a feeler gage at the upper end. This pipe is situated inside a larger piece of casing attached to a road box for surface protection. The lower end of the small pipe is attached to the top of the utility to be monitored and data is collected by determining whether the top seems to be moving downward. Unfortunately, such an instrument works well only if the monitored utility is exposed in a trench, and the inner pipe of the instrument is attached before the utility is re-covered with backfill. When such an installation is attempted with a utility that is not exposed, one of two things may happen: (a) because the location of utilities is seldom known with absolute certainty, there is a danger that the installing drillers may penetrate the utility, leading to a larger problem than the new tunnel under construction would have created; and (b) in the confines of a small drill hole it is extremely difficult to actually attach the monitoring pipe to the top of the utility, so it is possible for the utility to settle without there being an indication from the instrument of the movement's true severity. In a case such as this, the best fallback position is to install a Borros Point (Figure 5-14) or an SPBX beside and

to invert the depth of the utility. If ground movement is observed at that location, it may be an indication that excavation procedures need to be modified to contain a problem. Depending upon its size and stiffness, a utility may be able to bridge over a zone of disturbance and so be in no immediate danger, but the ground settlement of a certain magnitude can be an indication that the movement needs to be arrested before it does become serious.

6) Horizontal Inclinometers

Horizontal Inclinometers are simply inclinometers turned on their sides and the transducers in the probe (conventional instrument) or sensors (in-place instrument) mounted such that the sensitive axes are perpendicular to the length of the pipe (Figure 5-27). In this way, an inclinometer is measuring the vertical rather than the lateral movements of the instrumented structure. One use for a horizontal inclinometer is in the determination of settlement of a utility along a reach that requires continuous data not producible by the utility monitoring points or extensometers described above. Due to the difficulty of continuous access for monitoring, such an inclinometer installation is more likely to entail an in-place instrument that can be remotely read, but even here access may pose at least a minor challenge. If the utility is large and the flow of contained liquids can be controlled, then inclinometer casing may be strung and attached to the roof inside the instrumented structure. If the utility is too small for entry or the liquids cannot be controlled, then it would need to be exposed in a trench for instrument attached to the outside and then backfilled. In either case, arrangements would be made for wiring to be run to a datalogger for essentially real-time monitoring. The difficulty of access for installation is an obvious drawback, but when the need for monitoring is over, it should always be possible to salvage the expensive sensors for re-use.

If entry into the utility were possible for installation, then it should also be possible for recovery efforts. If the instrument were installed and then covered over by the backfill, a small manhole will have been provided for access to the reference head and the wiring, and it is from here that the sensors and their attached wires can be removed.

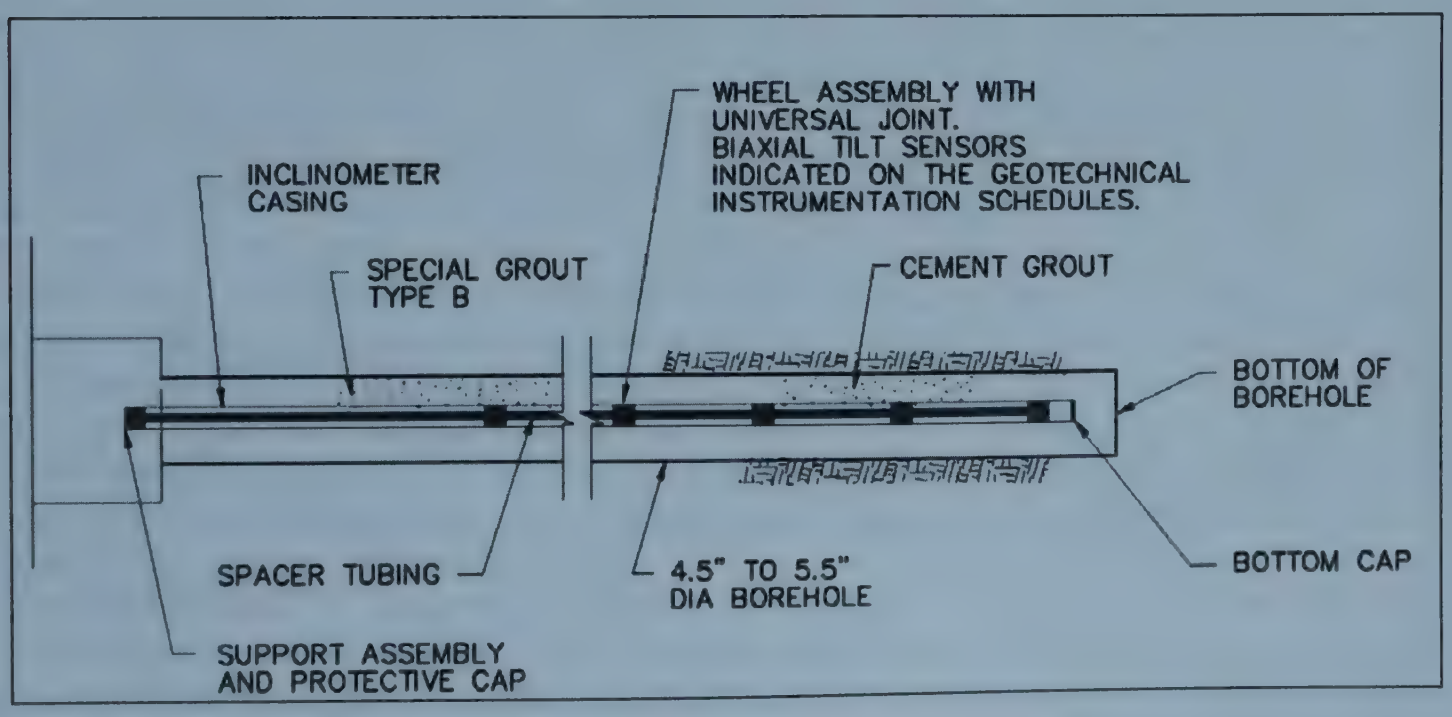


Figure 5-27: Horizontal In-Place Inclinometer[1]

7) Liquid Level Gages

Liquid Level Gages are systems of sensors installed in an array that measures the height of a column of water within each gage as shown in Figure 5-28. Sensor gages are connected by small 0.5 to 2.5 cm diameter tubes or pipes to a reference gage outside the zone of influence. The reference gage is actually a reservoir, with its contained liquid generally

kept under pressure to avoid the undesirable effects of barometric changes. The liquid completely fills all of the tubes throughout the array of components, none of the liquid is exposed to the outside atmosphere, and so it is referred to as a closed pressurized system. With the liquid always at the same elevation, settlements of the instrumented locations are indicated as the heights of the columns of water within the gages change in relation to the gage housings, which are moving. Signal outputs are most commonly driven by LVDTs (see description under electrical crack gages below) or vibrating wire force transducers. The closed systems are small and flexible and can be configured to fit into the convoluted layouts of many instrumented structures. Readings are collected remotely through the wiring of the system to a datalogger. Such systems are commonly installed in or on a structure where continuous settlement measurement to an accuracy of several millimetres is needed and where continued access for maintenance is not a large problem.

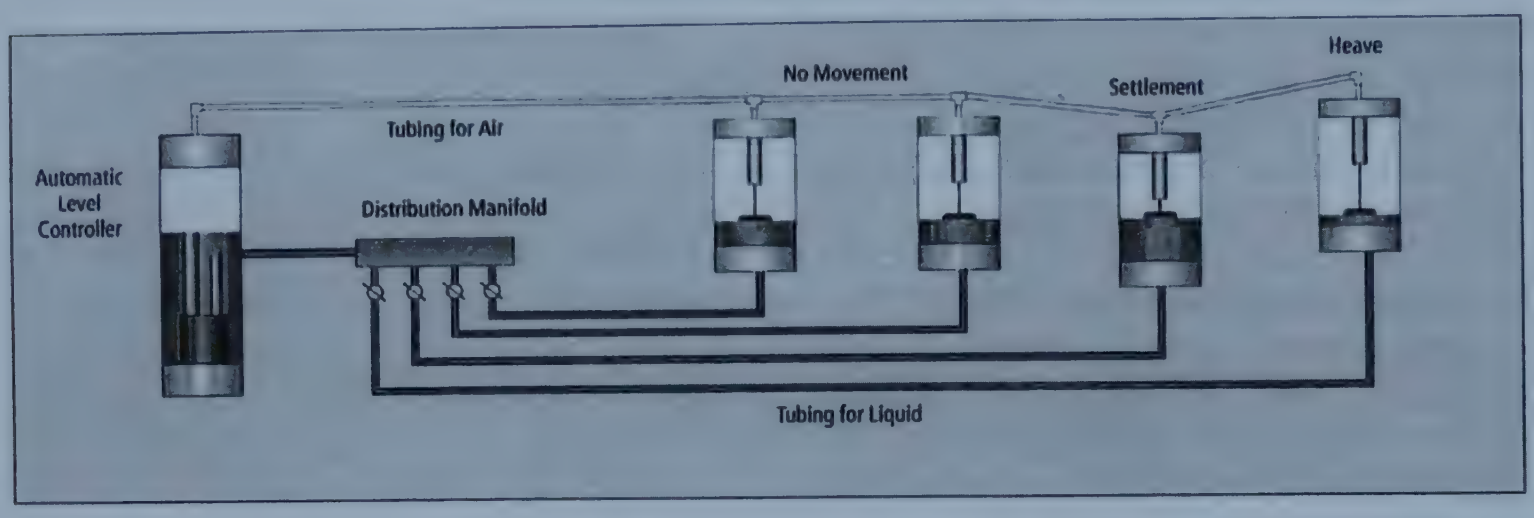


Figure 5-28: Multipoint Closed Liquid Level System[1]

Maintenance visits are a must with these systems, and so the issue of access has to be taken seriously. During installation, which must be performed with great care, the system has to be charged with de-aired water and then purged to make certain no air bubbles have intruded to remain within it. This is one reason most installations utilize some kind of semi-transparent plastic tubing; it permits visual detection of bubbles and makes purging them easier. This is critical because air bubbles will migrate to high points in the tubing or to the sensors themselves and can cause readings to be very inaccurate or can even shut down the system altogether. Then, during operation, it is very common for bubbles to appear in spite of careful installation. This may occur due to leakage from the outside, tiny amounts of air coming out of solution and accumulating, etc. Interestingly, the pressurization of the system can inhibit the emergence of bubbles but never stop it entirely. No closed system is immune to this problem and maintenance visits may be required for purging and de-airing as often as every 6 to 8 weeks. This is why continued access can be so important to the closed pressurized system's functionality. The maintenance problem can be largely overcome through the use of an open channel system which consists of sensors connected by pipes that are only half-filled with water as shown in Figure 5-29. Open to the atmosphere, neither the liquid nor the sensors are affected by the problem of air bubbles. They can be installed to lengths of several kilometres, operate for many months with hardly any maintenance, and still detect movements to sub-millimetre accuracy. However, such systems are large, heavy (due to the piping), sometimes difficult to install in structures with complicated layouts, and are much higher in front-end costs than the smaller closed systems.

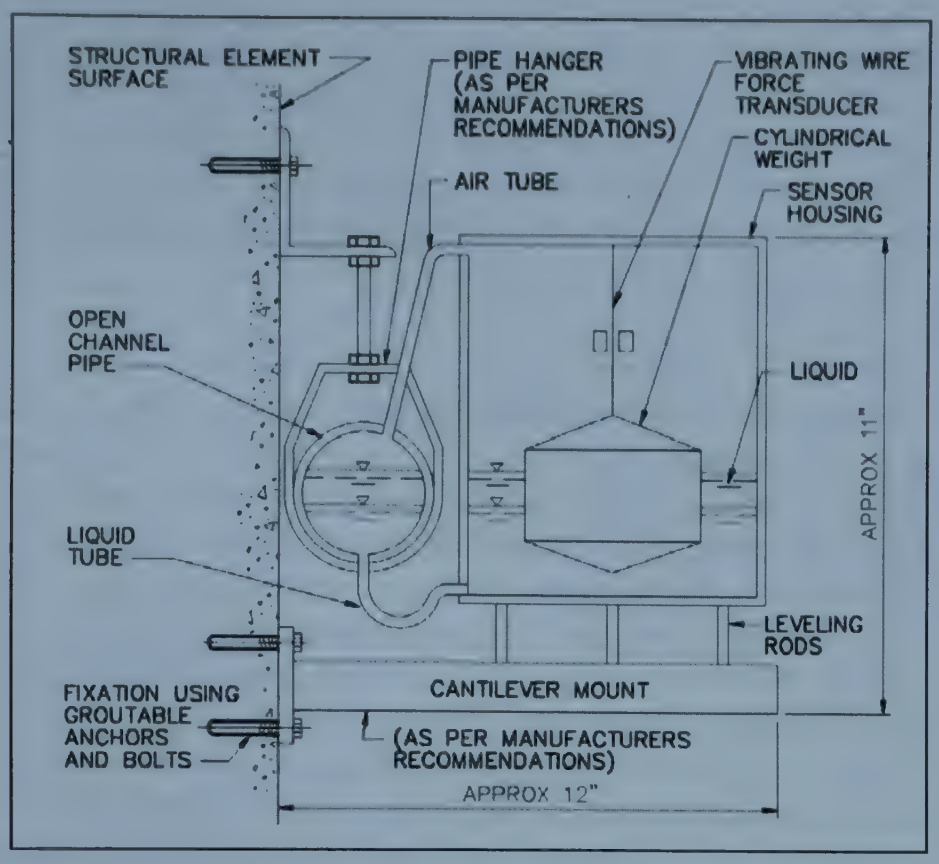


Figure 5-29: Open Channel Liquid Level System[1]

8) Tilt Sensors on Beams

Tilt Sensors on Beams, when packaged to monitor elevation changes rather than tilt per se, consist of sensors attached to metallic rods or beams, with the beams linked together with pivots (Figure 5-30). By monitoring changing tilt of each sensor and knowing the length of each +/- 5-foot long beam, users can calculate elevation changes of each pivot with respect to the datum. The relative tilt of each sensor and beam is set in the field and elevation change data is determined by making an initial scan of readings, called the reference set, and mathematically subtracting readings in that scan from each subsequent scan. All elevation change data is referenced to one end of the system defined as the datum. Ideally, the datum is in a stable area not likely to move, and its absolute elevation is generally determined by an initial optical survey. Integrating the data is an iterative process as settlements are computed from sensor to sensor. Readings are collected by having the system connected with a data logger for almost real-time monitoring.

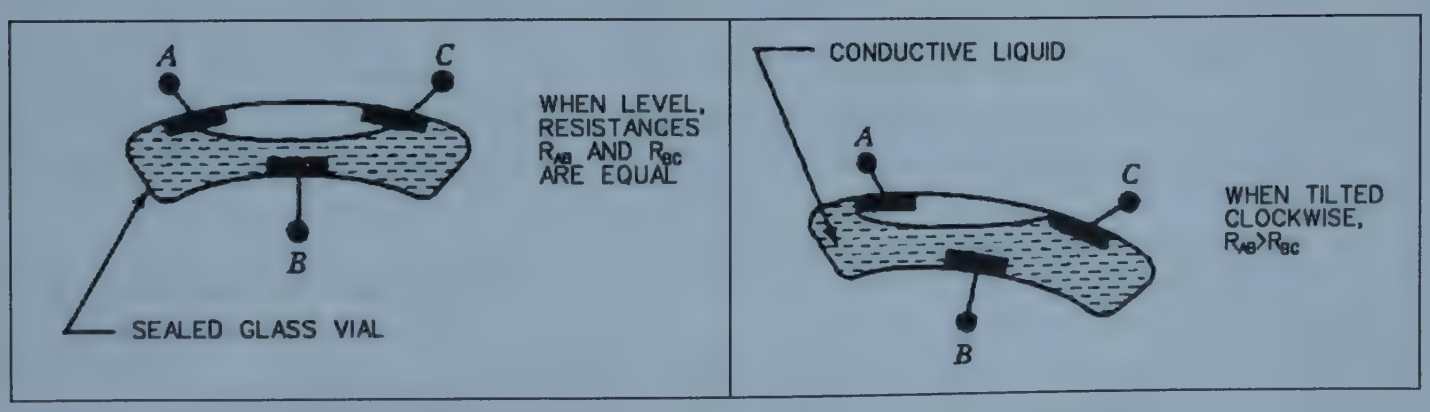


Figure 5-30: Schematic of Electrolytic Level Tilt Sensor[1]

Such installations can work on bridges, the balustrades of buildings, the walls or safety walks of existing tunnels, or even railroad tracks. However, they do depend upon sensing the mechanical movements of a string of components, and the components need to be as free from interference as possible. If installed where workers or moving equipment may be present, they have to be protected by the installation of metallic housings or half rounds of the heavy plastic casing. Another potential problem stems from changing

temperatures, especially in the outdoors where there may be exposed to severe or very changeable weather. Although the sensors may fare as well as they would in any other type of installation, such as in a tiltmeter housing, the beams, and the pivots are metal and subject to thermal effects with the potential to skew the data in unexpected ways. Users need to be aware that, if even one sensor or sensor/beam combination fails for any reason and requires replacement, the whole string of sensors and beams will need to be reinitialized.

9) Crack Gages

Crack Gages (also sometimes called Jointmeters) as installed on structures are typically used for monitoring cracks in concrete or plaster, or for determining whether movement across joints is exceeding a structure's design limits. The first appearance of cracks can be an indication of structural distress, and their growth, either in width or in length, can be an indication that stress is increasing, as can the continued widening of an expansion joint. There are several ways of measuring these movements; only the two most common can be covered herein. As shown in Figure 5-31, a Grid Crack Gage consists of two overlapping transparent plastic plates, one installed on each side of the discontinuity and held in place with epoxy or mounting screws. Crossed cursor lines on the upper plate overlay a graduated grid on the lower plate. Movement is determined by observing the position of the cross on the upper plate with respect to the grid. Data is kept in notebooks and has to be key punched into a computer if needed for an electronic database. Such gages are inexpensive to purchase and install, but readings may vary with changes in monitoring personnel and this has to be guarded against. There are three circumstances in which such simple devices may prove inadequate: (a) where cracks are too narrow or are widening too slowly for the human eye to detect their growth; (b) where continued physical access is very difficult and remote monitoring is required, and (c) where something close to real-time monitoring is required. Such difficulties may be overcome through the selection and installation of Electrical Crack Gages as shown in Figure 5-32.

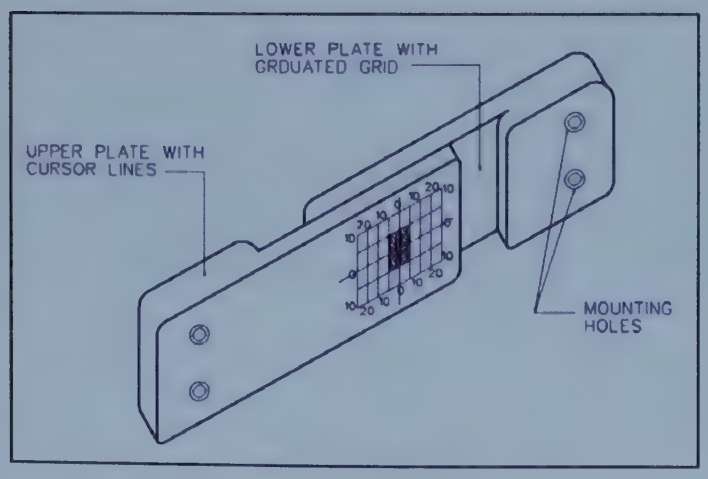


Figure 5-31: Grid Crack Gauge[1]

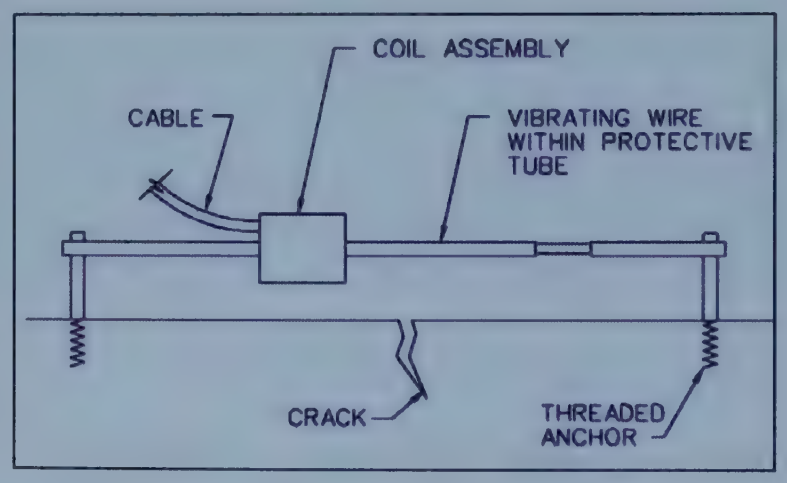


Figure 5-32: Electrical Crack Gauge[1]

There are a number of electrical gage types, but most are based on an arrangement of pins attached on opposite sides of a joint or crack, with the pins connected by sliding extension rods whose differential movements are detected by a built-in transducer. The most common transducer is the linear variable displacement transformer (LVDT) that consists of a movable magnetic core passing through one primary and two secondary coils. Data readouts depend upon the detection and measurement of differences between voltages generated in the secondary coils, magnitudes of which depend on the proximity of the moving magnetic core to the secondary coils. Users may prefer to pick up the gage signals by using a small low-power radio transmitter installed at the instrument location to avoid the transmission of alternating currents through long lead wires that can introduce output-degrading cable effects.

(3) Tunnel Deformation

When the temporary or permanent structural support for a tunnel is being designed, calculations are performed to predict the kinds of movements and stresses the support can safely be subjected to before there is a danger of failure. It is the job of instrumentation specialists to track those movements and stresses and provide guidance on whether the support or the construction process needs to be modified to ensure short-term safety and long-term stability of the completed tunnel. For braced excavations, it is standard practice to measure the loads on some of the support members, and often to combine these with measurements of the support member deflections if the measurement of ground movements outside the support system is not sufficient to present a complete picture of support performance. It is possible to thus monitor the significant performance-related behaviour of soldier piles, slurry walls, struts, tiebacks and other elements of open cut or cut-and-cover excavations. In mined tunnels, it is generally more common to use deflection measurements as the first line of defence against adverse developments because the eccentricities in the movements of many support members, such as steel ribs, make stress and load measurements much more complicated and prone to varying interpretation than they are for braced excavations.

Monitoring of the tunnel itself is similar to ground movement monitoring, using the following instrumentation:

- Deformation Monitoring Points
- Inclinometers in Slurry Walls
- Surface Mounted Strain Gages
- Load Cells
- Convergence Gages
- Robotic Total Stations

1) Deformation Monitoring Points

Deformation Monitoring Points (DMP) on support elements take several forms, but all have one thing in common: they are semi-permanent points to which a surveyor can return repeatedly and be certain of monitoring exactly the same point. A DMP may consist of a short bolt inside an expandable sleeve if mounted in a small drilled hole in concrete, such as a slurry wall (Figure 5-33), or may be the head of a bolt that is tack welded to a steel surface such as the top of a soldier pile. A DMP can be surveyed for both lateral and vertical movements to help determine whether the upper reaches of support may be "kicking in" or perhaps settling downward as the ground moves. If mounted in or on a vertical surface, the bolt head must have enough stick-out to permit a stadia rod to be rested on it. If mounted in or on a horizontal surface, the bolt head must be rounded, especially if it is to be used for determining vertical movements, for the same reason that a round head DMP is important in the monitoring of roads and streets. If the DMP were simply a flat plate, it would be too easy for the rod person to set up on a slightly different spot with each survey, especially if the monitored support element were bending inward, and this could result in cumulative errors in the elevation data plots. For support elements, it is desirable that elevation surveys be carried out to an accuracy of as little as 5 or even 2 mm, and every effort should be expended to make this as easy for the surveyors as possible. The largest problem for this type of monitoring is the same as was previously discussed in ensuring survey accuracy, except that the difficulties may be greater in this instance because the surveyors are more likely to be working in the middle of heavy construction activity, hence more rushed and/or more distracted.

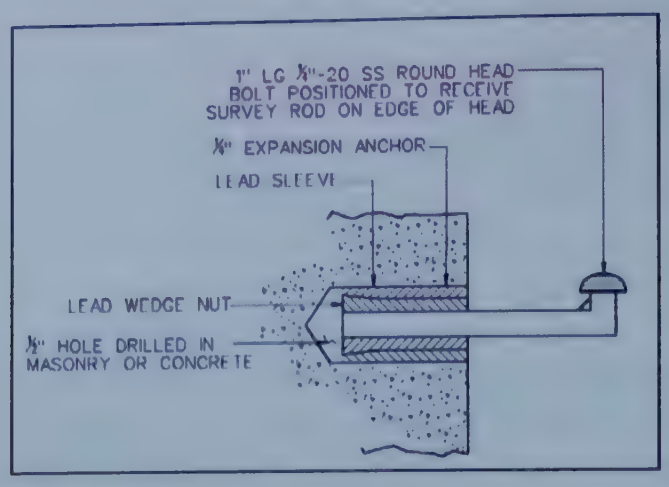

1" LG ⅜"-20 SS ROUND HEAD BOLT POSITIONED TO RECEIVE SURVEY ROD ON EDGE OF HEAD
⅜" EXPANSION ANCHOR
LEAD SLEEVE
LEAD WEDGE NUT
½" HOLE DRILLED IN MASONRY OR CONCRETE

Figure 5-33: Deformation Monitoring Point in Vertical Masonry or Concrete Surface[1]

2) Inclinometers in Slurry Walls

Inclinometers in slurry walls are very similar to those previously described for ground installations, except that drilling is not generally required (Figure 5-34). Installation is accomplished by fastening the instrument casing inside the wall panel's rebar cage as that element is being fabricated. As the cage is lowered into the slurry trench, the inclinometer casing goes with it and remains in place as the slurry is displaced during the introduction of concrete. Because the slurry wall will have been designed to penetrate below any zone of expected movement, the bottom of the inclinometer casing is the presumed unmoving reference from which tilting of shallower points along the casing is calculated. Monitoring is accomplished by the instrumentation specialist lowering a probe to the bottom of the casing and collecting readings as it is winched back to the surface. The biggest problem with an inclinometer in such an installation is the essential impossibility of repair if anything has gone seriously wrong. Also, one cannot replace the instrument by simply drilling a new casing into reinforced concrete a foot or two away. If the instrument is considered absolutely essential, it might be feasible to drill a new one into the ground just in the back of the wall, but long drill holes tend to wander away from the vertical - perhaps in a direction away from the slurry wall - and chances are not good that the replacement instrument would truly indicate what the slurry wall itself is doing. This possibility of damage is one argument against the installation of in-place inclinometers in this type of support. Depending on the seriousness and the depth of any damage to the casing, some or most of the expensive sensors could be stuck and impossible to recover.

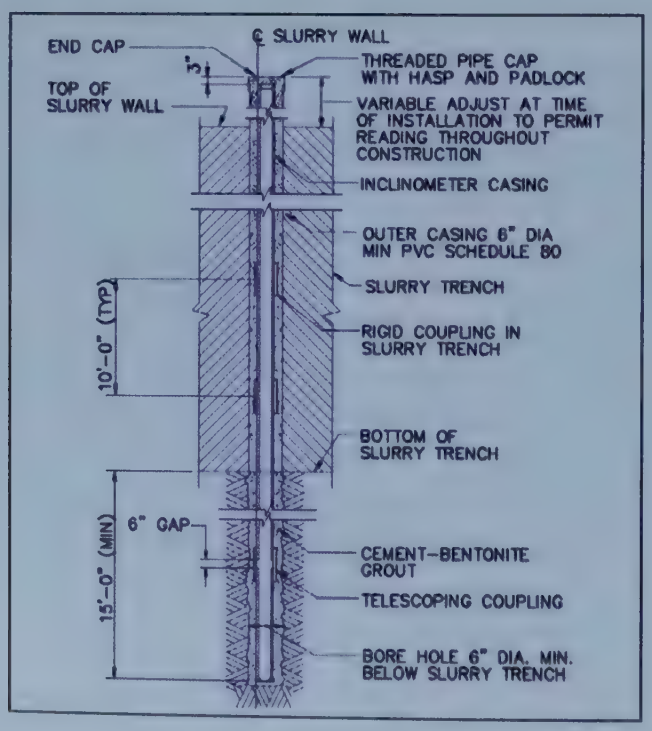

℄ SLURRY WALL
END CAP
TOP OF SLURRY WALL
THREADED PIPE CAP WITH HASP AND PADLOCK
VARIABLE ADJUST AT TIME OF INSTALLATION TO PERMIT READING THROUGHOUT CONSTRUCTION
INCLINOMETER CASING
OUTER CASING 6" DIA MIN PVC SCHEDULE 80
SLURRY TRENCH
10'-0" (TYP)
RIGID COUPLING IN SLURRY TRENCH
BOTTOM OF SLURRY TRENCH
6" GAP
15'-0" (MIN)
CEMENT-BENTONITE GROUT
TELESCOPING COUPLING
BORE HOLE 6" DIA. MIN. BELOW SLURRY TRENCH

Figure 5-34: Inclinometer Casing in Slurry Wall[1]

3) Surface Mounted Strain Gages

Surface Mounted Strain Gages are most commonly used to determine stresses and loads in struts across braced excavations. Although many kinds are available, the vibrating wire type finds the widest application because of a stable output that is in the form of signal frequency rather than magnitude. Figure 5-35 shows a schematic of the vibrating wire-type strain gage. In this instrument's packaging, a length of steel wire is clamped at its ends inside a small housing and tensioned so that it is free to vibrate at its natural frequency. The frequency varies with the tension, which depends upon the amount of compression or extension of the instrumented strut to which the gage has been attached by spot welding or bolting. The wire is magnetically plucked by a readout device, and the frequency changes are measured and translated into strain, which can, in turn, be translated into stresses and loads on the instrumented member from a knowledge of the material's modulus. The point of the measurements is that designers will have calculated the permissible loads in the struts and the instrumentation specialist is collecting data to determine if the struts may be approaching their design limits. Gages are typically mounted 2 to 3 strut widths/diameters from the ends in order to avoid the "end effects" that degrade accuracy. Because a strut will bend downward from forces of gravity even when not under load, creating compression at the top and extension at the bottom, it is necessary to install several gages arranged in patterns around the neutral axis and average the readings for the closest possible approximation of maximum stress.

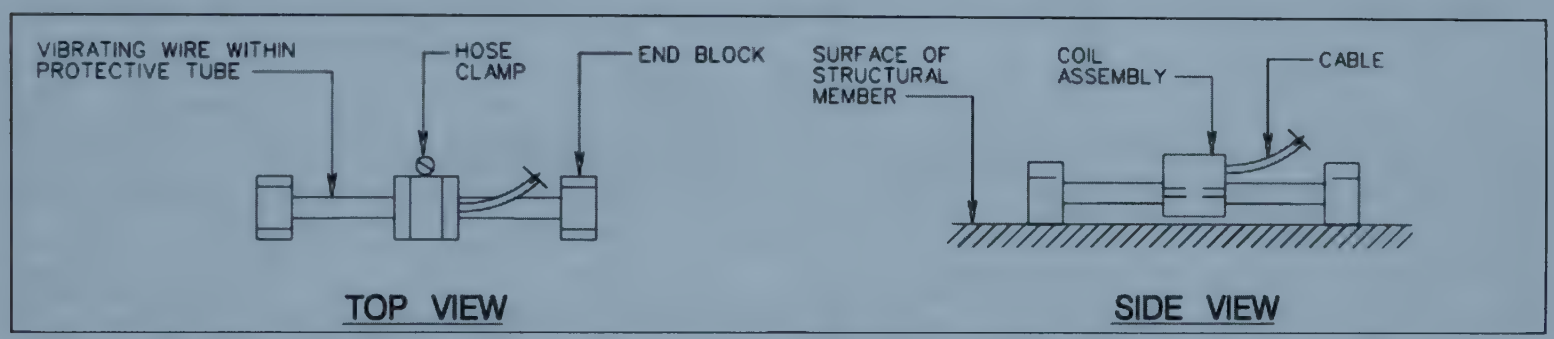


Figure 5-35: Surface Mounted Vibrating Wire Strain Gauge[1]

Many things can go wrong with such installations, and they need to be undertaken with the greatest of care by experts with good experience. However, as noted in the introduction, the greatest problem with these types of measurements can reside in the agendas of the various parties who may need to understand the data and perhaps take action to mitigate apparent problems. Measurements of ground and structure *movements* are in general understood by most people associated with tunnelling. However, stresses and strains require a certain amount of sophistication to comprehend, and even among those with sophistication, interpretations of what the data mean can vary wildly. It is very common for constructors and their consultants to believe instruments are faulty, that data has not been properly collected, or data has not been properly reduced to good engineering values if taking mitigative action is going to interfere with the field operations. Also as previously noted, this is why the use of strain gages can be fraught with complications if used on the steel ribs in mined tunnels. Compared with struts in braced excavations, ribs under load can bend and twist in many unanticipated ways, and placing strain gages in the best configurations just where they need to be placed can be difficult.

4) Load Cells

Load Cells are, in general, arrays of strain gages embedded in housings which are placed in instrumented tunnels under construction in such a way that loading forces pass through the cells. For the reasons stated in the strain gage description above, very stable vibrating wire transducers are the data collecting elements on which most load cell configurations

are based. As shown in Figure 5-36, the load cell is a "donut" of steel or aluminium with several transducers mounted inside in a way to be read separately and averaged in the readout device. Transducers are oriented so that half of them measure tangential strains and half of them measure axial strains. Integration of the individual strain outputs helps reduce errors that might result from load misalignment or off centre loading. Although load cells may be installed on tensioned rock bolts in mined tunnels, their more common use is in non-braced open excavations. Here the cell is installed on a tieback near the rock face and locked down with thick bearing plates, washers, and a large steel nut. In most cases, the instrument will be wired for electrical remote reading because it will be left in place for a considerable amount of time, and direct access for data collection will often not be available once the excavation has passed below the tieback's level. If a load cell seems to be producing questionable data, the most likely cause is a misalignment of the instrument on the shaft of the tieback. For the most part, tiebacks are angled downward rather than being installed horizontally, and careful placement of bearing plates and washers of the correct thickness is essential.

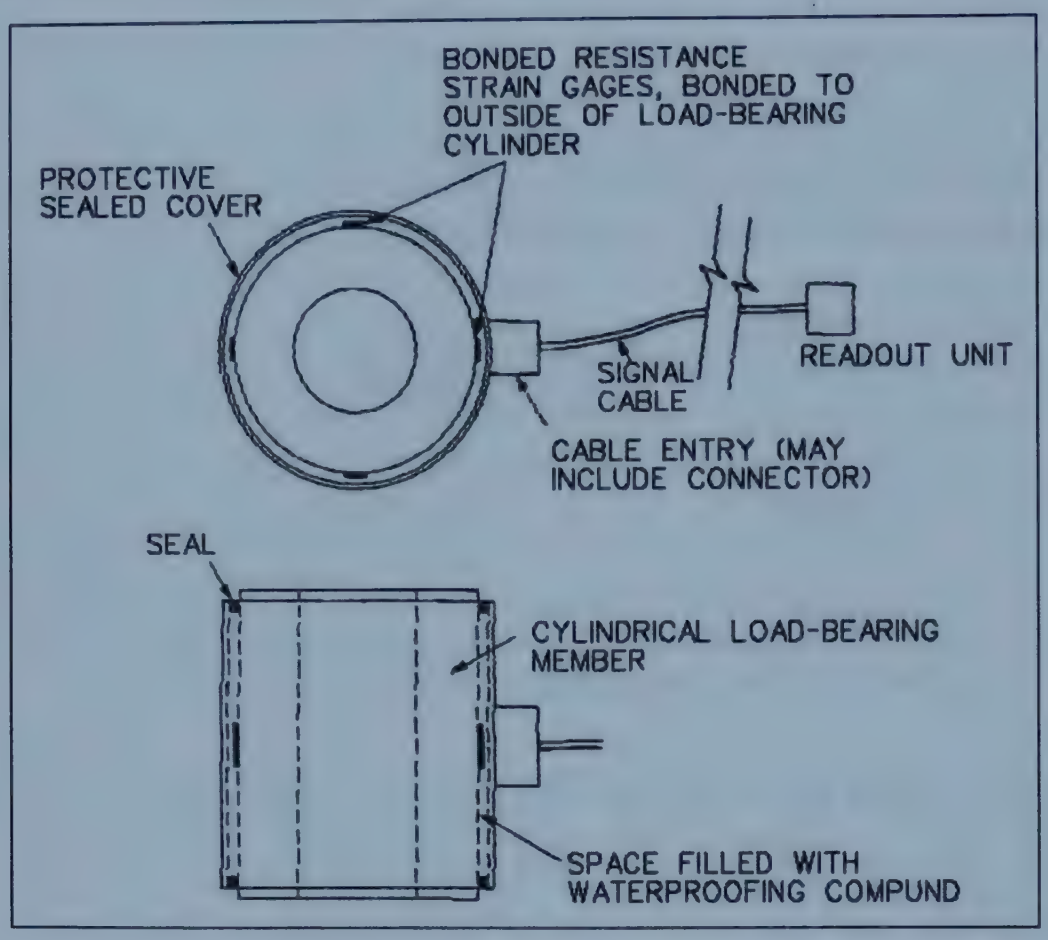


Figure 5-36: Schematic of Electrical Resistance Load Cell[1]

5) Convergence Gages

Convergence Gages may be used on tunnel supports just as they are in the monitoring of tunnelled ground. For the most part, it is best to monitor the ground itself because that gives the best from-the-beginning measurements that constitute good initial movement readings. However, if it is necessary for whatever reason, similar anchors, eyelets, cradles, and survey targets can also be installed on steel ribs, shotcrete linings, and final concrete linings. As in the earlier discussion, it appears that distometers should be the chosen replacement for the older tape extensometers when measuring the distortions. In modern mining, there are situations which do not lend themselves to the easy measurement of ground movements from the tunnel itself because of the chosen method of ground support. The most common of these situations result from the use of a TBM where pre-cast concrete segments are erected after each push to form another 1.2 or 1.5 m of completed tunnel ring directly behind the shield. As with monitoring of ground movements, the most common problem with these measurements of lining distortion is the difficulty of getting good lines of sight directly behind the machine in order to achieve a true zero movement initial reading.

6) Robotic Total Stations

Robotic Total Stations as described for existing structures in "(1) Ground Movements - Vertical & Lateral Deformations" can also be used to monitor the opening that is under construction. However, there are possibly more limitations on underground installations than on installations associated with inhabited buildings above. A total station instrument sitting atop its motorized support platform has a footprint of at least one square foot, its height is a bit greater, and the platform may protrude from the tunnel wall as much as 50 cm. The package would hardly fit well into a small tunnel and would be constantly on the move as the tunnel advanced. Hence, the most logical place for such monitoring of active construction would be within a large mined chamber or perhaps a large open excavation. Even here, however, the uses might be more restricted than is at first obvious. The average construction site is a hostile environment, and the decision to install such an expensive piece of equipment cannot be taken lightly. The dust alone on some construction sites might be enough to force heavy maintenance procedures on the part of users. Even in the outdoors, target prisms have to undergo regular maintenance because signals can be so degraded by the accumulating dust from the atmosphere. The interior of a construction site is much worse; maintenance of the expensive instrument itself would be more onerous than usual, and many target prisms would likely be at a height that requires the use of a man lift for access. It seems probable that the best use for robotic total stations would be found in an advanced stage of large construction where most of the final concreting has been accomplished and the structure needs to be monitored in something close to real-time as the finish stage of construction proceeds.

(4) Dynamic Ground Movement - Vibrations

As opposed to the measurements discussed earlier, which concerned long-term effects of the construction of a tunnel on the gross movement of either the ground or buildings adjacent to the tunnel, these measurements are taken to establish the potential impact of drill and blast excavation on structures. The use of explosives often causes concern on the part of stakeholders in the neighbourhood of tunnel excavation. Aside from the images generated by blasting, there is a real concern due to the sudden (and sometimes perceptible) motion generated by the explosive energy that is not used in fragmenting rock, but that propagates away from the blast site. The usual method of monitoring these motions is based upon research studies that correlate the potential for damage from blast vibrations with the motion of the ground

There are two general types of equipment used for monitoring the Dynamic Ground Movement induced by blasting:

- Blast Seismographs
- Dynamic Strain Gages

Blast seismographs are used to monitor ground motion at structures within the zone of influence. Dynamic strain gages are used to monitor the actual strain (or relative displacement) of structural elements of such structures. Both of these instruments monitor data during the actual blast event, though for convenience they may be set to monitor before the actual blasting.

1) Blast Seismographs

The standard blast monitoring equipment has been blast seismographs. A blasting seismograph is a specialized instrument which measures and records the ground and air vibrations from a blast. A seismograph has two sensors: a geophone (for monitoring ground waves) and a microphone (for monitoring airwaves). This is the dynamic measurement of a wave that is extended in time and space; therefore, there is no single value that totally describes a blast wave. Through many years of research, it has been determined that the single most descriptive value that can be associated with the potential for structural

damage is "Peak Particle Velocity," or PPV. As a blast vibration wave travels, it is analogous to waves on water. If one imagines a bobber on the water, the velocity of the bobber moving as the wave passes is the particle velocity. The peak particle velocity is the highest value of velocity during that wave passage. Blast seismographs measure three components of ground motion: vertical, longitudinal (horizontal along the direction from the blast), and transverse (perpendicular to that direction). The highest of these three values is used as a vibration criterion. There is typically a fourth channel used for above-ground blasting that monitors air overpressure or air blast, but this channel is generally not used when blasting in tunnels, since there is no direct exposure to surface structures. As mentioned, criteria for blasting have been developed based upon occurrences of damage. Most of the studies done have concentrated on typical residential wood-frame structures. Because structures respond in many ways to vibrations that are imposed at the base of the structure, in most cases the vibration is monitored on the ground outside of the structures. The potential for damage is then inferred from the association of the PPV with the potential for damage of a particular structure type. Sometimes the frequency of the vibration is also incorporated in the criteria, but this is not always the case. Criteria are usually adjusted upwards when the structure type is more substantial or engineered, relative to the criteria used for residential structures.

2) Dynamic Strain Gages

Because there is so little accumulated damage data for some structures, an alternative method for monitoring, using dynamic strain gages, has been adopted recently. For engineered structures and infrastructure elements, actual failure criteria can be developed that is independent of the mode of excitation. In this case, a level of strain, which is a dimensionless measure of relative motion, is used as a criterion for the avoidance of damage. Strain ε is defined as$\varepsilon = \Delta l / l$, where Δl is the change in length of an element, and this is divided by the length of the element. Measurement on a small length of a structural element may then represent the deformation of the entire element when the total structural configuration is known. Dynamic strain gages are traditionally thin foil resistance gages, which are connected to other gages in what is called a Wheatstone bridge. The gages change resistance when they are deformed. This arrangement of gages will then produce a voltage output that is monitored during the blasting process. The foil gages have been in use for over half a century, initially in static strain environments. Though it is a mature technology, there are sometimes problems when the gages are in electrically noisy environments, or where there are temperature fluctuations. Although they have only been used recently, piezoelectric and fibre optic strain gages are not susceptible to as many problems as are the foil gages. Dynamic strain gages, since they measure strain on a particular element that is of concern, must be carefully located to obtain the values that can be associated with the potential failure of the element. Strain gage mounting must be carefully chosen on a representative location, and a measurement on the ground surface (as is done with blast seismographs) is not appropriate. There is not as much background documentation in associating damage with strain from blasting; however, the fundamentals of strain-based failure criteria have been used for many years. The use of strain gages is limited to where there is a sound understanding of the actual limiting strain values that can be accepted as safe, based upon engineering documentation.

(5) Groundwater Behaviour

Three standard types of instrumentation are used to determine the effect of tunnel construction on groundwater movements and pressures:

- Observation Wells
- Open Standpipe Piezometers
- Diaphragm Piezometers

1) Observation Wells

Observation Wells are the simplest and least expensive instruments in the list of devices used to determine groundwater pressures. A well consists of a perforated section of pipe attached to a riser pipe installed in a borehole filled with filter material, generally sand or pea gravel (Figure 5-37). The filter prevents fines from migrating in with the water and clogging the well. The filter may extend to only 1 to 2 m above the perforated section or may go almost to the ground surface, but the well must have a mortar seal near the top of the riser pipe to prevent surface runoff from entering the hole. Also, a vent is required in the top cap so that water is free to rise and fall in the pipe. The height of the groundwater table is generally measured by lowering an electrical probe at the end of a graduated cable until it touches the top of the water. A circuit is then completed and so indicated by the flicker of indicator light or sound of a buzzer at the upper end of the cable. Such wells are installed in tunnelled ground where it is assumed that the ground is continuously permeable and groundwater pressures will increase uniformly with depth. Tunnel designers try to gain an understanding of the groundwater regime as design proceeds and often will specify the level to which the water must be pulled down by a dewatering program before construction is permitted to proceed too far. It is common to require dewatering to a level 1 to 2 m below the final invert for either a soft ground mined tunnel or braced excavation. An observation well would then be installed to 0.5 or 1 m below that drawdown level to be certain of detecting the new during-construction top of the water table. The most common problem with observation wells is that they may not be the instrument appropriate for the situation because the complexity of geologic stratification is actually greater than anticipated. If readings seem inexplicable, it may be because the water table corresponds to the head in the most permeable zone rather than to a straight-line correlation with depth from the ground surface. It is possible that the wells may need to be supplemented with other instruments such as piezometers.

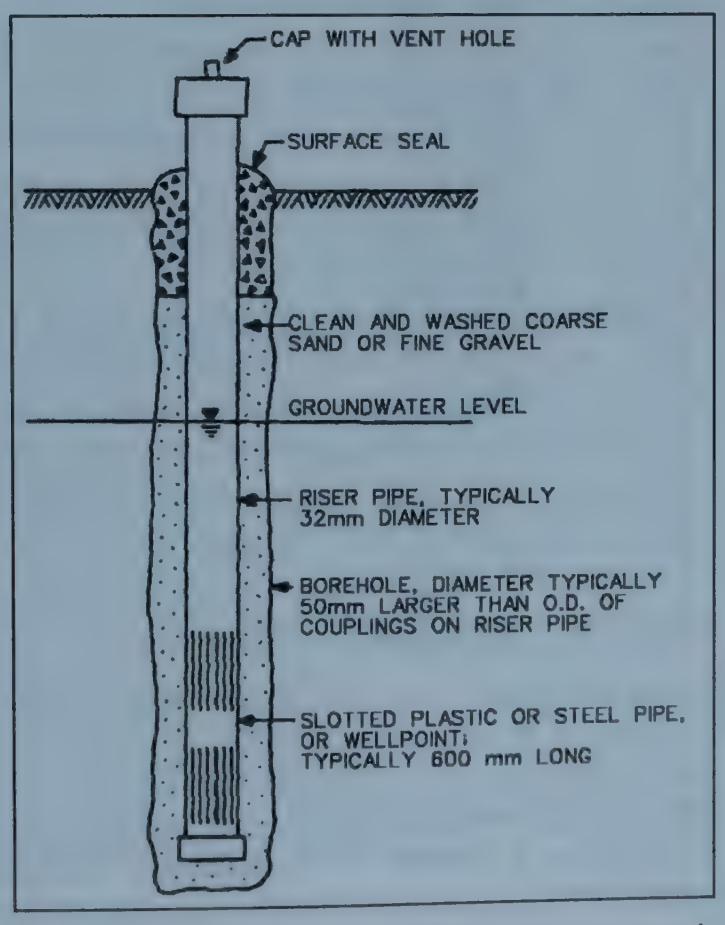

CAP WITH VENT HOLE
SURFACE SEAL
CLEAN AND WASHED COARSE SAND OR FINE GRAVEL
GROUNDWATER LEVEL
RISER PIPE, TYPICALLY 32mm DIAMETER
BOREHOLE, DIAMETER TYPICALLY 50mm LARGER THAN O.D. OF COUPLINGS ON RISER PIPE
SLOTTED PLASTIC OR STEEL PIPE, OR WELLPOINT; TYPICALLY 600 mm LONG

Figure 5-37: Schematic of Observation Well[1]

2) Open Standpipe Piezometers

Open Standpipe Piezometers are very similar in construction to observation wells, with one major difference: the porous filter element is sealed with bentonite grout into a particular permeable stratum so the instrument responds to groundwater pressure only at

that level and not to pressures at other elevations (Figure 5-38). Such a piezometer may be installed in soil strata or in bedrock and will function as long as the porous intake and filter are sealed in a zone that permits water to flow. In soil, the instrument will be measuring pore water pressure; in rock, it will generally be measuring joint water pressure. The instrument creates little or no vertical hydraulic connection between strata and, in contrast to simple observation wells, readings will be more accurate. If stratification is somewhat complex, several piezometers installed at different depths in the same small area would probably reveal more than one level of pressures, as in the case of a perched water table above a clay stratum exhibiting pressures different from those in a permeable stratum below the layer of clay. In construction monitoring, it is usual to install the porous intakes at the critical levels only, as in just below the inverts to where the water table needs to be lowered. Another common depth for the intakes would be at the boundary between an upper layer of sand and a lower layer of impervious clay in which the excavation bottoms out. In the latter situation, the dewatering subcontractor would probably be able to pull the water table down only to 1 - 2 m above the clay, and that is the elevation that would need to be monitored. Lack of expected response from an open standpipe piezometer is sometimes caused by clogging of the filter due to repeated water inflow and outflow. This may be remedied by high-pressure flushing, something readily accomplished if the drill rig used during installation is still in the area. A more serious problem would result from the porous intake having been installed in relatively impermeable silt or clay stratum because the borehole was not properly logged prior to installation. The only solution would probably be to install another instrument - perhaps another type of instrument - at the same plan location, with more attention being paid to good geologic logging and placement of the porous intake.

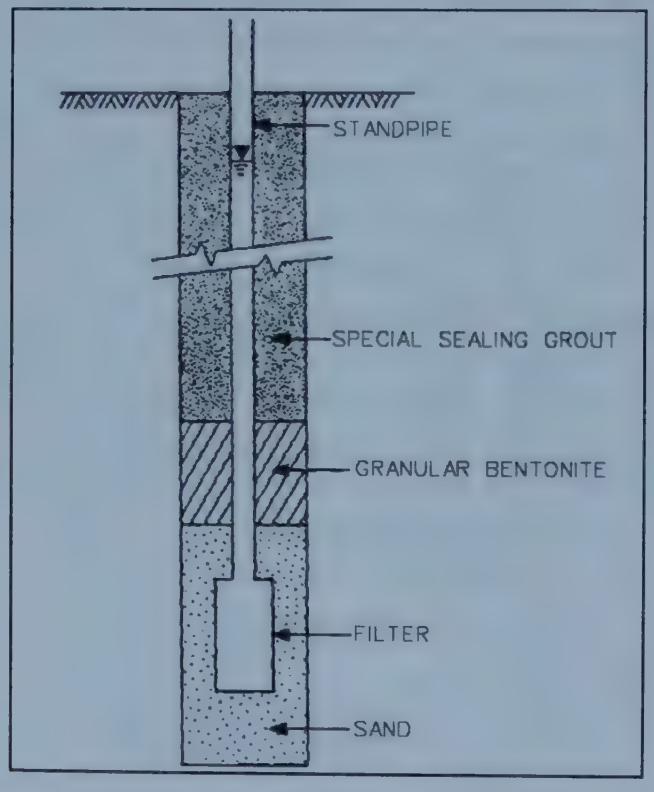


Figure 5-38: Schematic of Open Standpipe Piezometer Installed in Borehole[1]

3) Diaphragm Piezometers - Fully Grouted Type

As noted earlier, a *piezometer* is a device that is sealed within the ground so that it responds only to groundwater pressure around itself and not to groundwater pressures at other elevations. There are several situations that point to the need for a device that is more sophisticated than the simple open standpipe instrument: 1. Need to measure pore water or joint water pressure in a stratum of very low permeability. The hydrodynamic time lag for an open standpipe instrument is large, meaning that it responds slowly to changes in the piezometric head because a significant volume of water must flow to register a change. This cannot happen in materials of low permeability such as clay or

massive bedrock with few joints. 2. Some situations make it undesirable to have a rigid standpipe connecting with the surface, especially in the midst of heavy construction. 3. Repeated water flow reversals can cause the sand or pea gravel filter to clog. 4. In very cold climates there is a chance of freeze-up and resultant loss of opportunity to collect data. 5. A large number of readings and/or something close to real-time monitoring may be required, but the open standpipe instrument does not lend itself readily to this type of data collection. Thus, there are times when monitoring personnel is forced to choose a type of piezometer consisting of a unit that is pre-manufactured to interpose a diaphragm between the transducer and the pressure source. Pneumatic, electrical resistance, and vibrating wire are the three most common types of such instruments. The vibrating wire types us usually chosen because it operates with a short time lag, offers little interference to construction, and the lead wires can easily be connected to a surface readout unit or to a datalogger for real-time monitoring. Even these instruments, however, have always suffered from a major shortcoming: the assumed need to place filters around the sensing units and granular bentonite/cement grout seals and backfilling in the boreholes around and above the monitored elevation. Bridging and material stickiness can make proper emplacement difficult and may lead to degradation of data accuracy or outright instrument malfunction. This emplacement difficulty particularly complicates the installation of multiple piezometers in one borehole, so if readings from various elevations are required, it may mandate the drilling of a separate hole for each elevation that requires measurement. An obvious way around these difficulties would seemingly have been to forgo the filters and encase diaphragm piezometers and their accoutrements in a cement-bentonite mix seal all the way to the surface in *fully-grouted* installations. However, the prevailing opinion for many years was that the grout around the sensing unit might have extremes of permeability that would prevent an instrument from responding accurately to changes in pressures. However, from work that began in 1990, it has now been shown that this does not have to be the case. A diaphragm piezometer generally requires only a small flow to respond to water pressure changes, and the grout is able to transmit this small volume over the short distance that separates the sensing unit from the ground in a standard size borehole. The response can be enhanced if the installer minimizes this distance, which can be accomplished with an expandable assembly that lessens the distance between sensor and borehole wall, thus reducing the thickness of the grout between sensor and ground. Studies have shown that accuracy of pressure measurements will be good not only when the permeability of the grout is lower than that of the surrounding ground (which had been assumed all along), but also when the permeability of the mix is up to three orders of magnitude greater than that of the surrounding ground. Obviously, every situation requires that some work be done to formulate a grout mix of an appropriate permeability to be effective at the site being monitored. Fully-grouted piezometers can be emplaced by loose attachment then detachment from a sacrificial plastic pipe that is withdrawn (along with any support casing) as the grout is tremied in from the bottom up. It is relatively easy to install more than one instrument in the same hole for water pressure measurements at several elevations. As many as ten in holes penetrating to 150 m depths have been successfully installed.

Good experience in a greater than 15-year time frame prior to 2009 has shown that most diaphragm piezometers need to be installed as fully-grouted types for the sake of increased simplicity and the collection of much more data at a lower cost than had been the case with older methods.

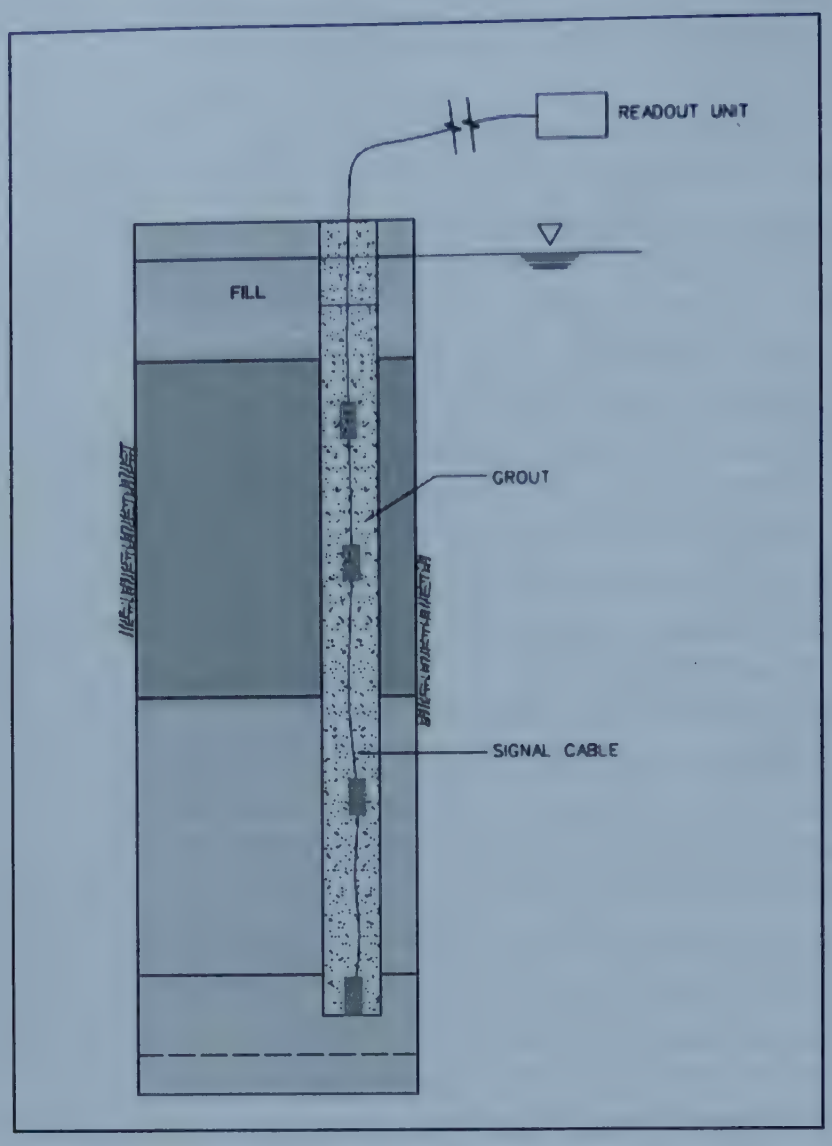


Figure 5-39: Schematic of Multiple Fully-Grouted Diaphragm Piezometer[1]

A continuing use for piezometers and observation wells depends upon their being left in place after construction is complete because of the effects the permanent structure may have on the groundwater regime. For example, if the water table remains depressed due to leakage into the new tunnels, a continuation in monitoring may indicate whether attention needs to be paid to the protection of wood support piles that remain exposed to air, or perhaps to wells or ponds that have been wholly or partially dried up. An opposite problem may stem from the mounding up of groundwater because its normal gradient is interrupted by the presence of the new tunnel, which may result in situations such as once dry basements that are now prone to flooding. Although leaving the instruments in place may result in increased maintenance costs, they can prove to be valuable sources of data when certain long-term problems are being investigated.

CHAPTER 6 SAFETY AND HEALTH

6.1 General

The safety and health of workers are essential in civil engineering work including tunnel construction. This includes the safety and health management of the worker as well as providing a comfortable working environment. In order to ensure the safety and health of workers in the tunnel, structured management, education, and supervision must be undertaken.

The duties of the supervisor (site foreman) include planning of machineries installation and construction schedule, disaster reporting, and others. In the construction work, it shall not only comply with the standards of the relevant laws and regulations but also create internal regulations to conform to the actual situation of the construction work. The installation of necessary equipment, organization of management, safety and health education to workers, regular inspections, and improvement shall be undertaken to prevent disasters.

The effective measures of safety and health of workers are to be considered from the planning stage of the design and construction plan considering the causes of the expected disasters during the construction.

The contents of this chapter are based on Japanese safety and health standards and regulations. The standards and relevant regulations of India shall be prioritized for tunnel construction in India.

6.2 Lighting

There are two types of lighting which are temporary light (point lighting) for the lighting for the places where the work is directly undertaken and long-term light (wide lighting) for an area such as passages where the work is not directly undertaken.

It is necessary to secure sufficient illuminance for the lighting of places where the work is directly undertaken such as tunnel face so that the work can be undertaken safely and efficiently. The required illuminance is 70 lux or more is preferable for such an area. In the inspection removal of the floating rock, the illuminance of the light shall be further increased by adding mobile lighting equipment. It is necessary to consider the contrast between light and dark as much as possible and to avoid glare when using these luminaires. In addition, it is necessary to perform sufficient maintenance and inspection of the mobile lighting equipment as it is liable to be damaged by frequent movement. The installation of waterproof protection on this equipment is highly recommendable.

In the passage section, necessary lighting must be installed to ensure the safety of worker traffic and the safe operation of vehicles. It is difficult to maintain a uniform illuminance over the entire passage though it is preferable to secure about 20 lux or more even in the darkest area. As for the fixed lighting, it is necessary to consider the durability and perform sufficient maintenance and inspection as it is usually used for a longer period. If the distance of the portals and tunnel face exceeds 100m, it is necessary to prepare an emergency guidance light (about 40W) in intervals of 40~50m which can light for a certain period by battery power in the case of power failure.

The illuminance may be insufficient partially in the tunnel due to dust, soot, and fog generated during the work or the voltage drop. Therefore, these matters shall also be considered in the lighting in the tunnel construction. In addition, warning signs shall be installed in area that is particularly dangerous.

6.3 Ventilation

During the construction, it is necessary to remove noxious gases, dust, and soot generated from the construction work or naturally from the earth as much as possible. Therefore, the installation of the appropriate ventilation system or equivalent measures shall be implemented to ensure a safe and hygienic working environment.

The main causes of the deterioration of the working environment inside the tunnel are as below;

1) Dust from drilling, blasting, stacking, spraying
2) Soot and fumes from exhaust of diesel engines
3) Noxious gas from organic solvent
4) Natural generated flammable, noxious gas and oxygen-deficient air
5) High humidity and temperature

The items of the ventilation can be classified as below;

1) Flammable gas: Methane (CH4) etc
2) Noxious gas: Carbon Monoxide (CO), Nitrogen Oxide (NOx), etc
3) Inhalable gas, Soot
4) Oxygen-deficient air
5) High humidity and temperature

During the planning of ventilation, the calculation of the ventilation volume and selection of the ventilation system shall be considered from the preliminary survey results of the items above along with cross-section, length of the tunnel, drilling and reinforcement method, equipment, and work schedule.

6.3.1 Noxious Gases

Table 6-1 shows the amount of noxious gas generated by the use of explosives. The worker shall not approach the blasted site unless the dust due to blasting has been appropriately diluted. Noxious gases emitted from the diesel engines of machineries vary depending on the usage conditions. Table 6-2 shows the machineries that require the use of equipment with emission control measures. Machineries according to the third standard value in Table 6-3 is specified as the standard value of the exhaust gas. It is necessary to calculate the amount of air that can sufficiently dilute the concentration of the noxious gases of the explosives and diesel engines as well as the dust of spraying. It is preferable to determine the diameter of the air tube, the capacity of the blower, etc. in consideration of the ventilation and mechanical efficiency along with the air volume.

Generally, the gasoline and LPG (Liquefied Petroleum Gas) engines shall not be used for tunneling work. The machineries with an internal combustion engine or a stove shall not be used in the area where the natural ventilation is insufficient. However, in case of unavoidable use due to the nature of the work, in other to the ventilation, the operator may select machinery which generates less carbon monoxide. In addition, the accidents such as the collision of the machineries may cause the leak of the highly volatile gasoline or LPG which lead to secondary disaster such as fire and explosion.

Table 6-1: Volume of Generated Noxious Gas (Reference Value)

Explosive	The volume of Noxious Gas*	
	CO (m^3/kg)	NO_x (m^3/kg)
Enoki-Dynamite No.2	8×10^{-3}	1.5×10^{-3}
Water Gel Explosive	5×10^{-3}	1.5×10^{-3}
Other dynamite	11×10^{-3}	2.5×10^{-3}
ANFO (Ammonium Nitrate Fuel Oil Explosive)	30×10^{-3}	20×10^{-3}

* Shaded sections need to be ventilated when each type of explosive is used and permissible concentration is CO: 50ppm, NO_x: 25 ppm.

Table 6-2: Machineries with Emission Control for Tunnel Construction

Types of Machines	Note
Backhoe	Applicable only for the construction machines with diesel engines (Output; 30 ~ 260kW (40.8 ~ 353PS)). Provided that the same shall not be applied to the vehicles with valid automobile inspection certificates which gas emission standards are designated in the safety standard.
Heavy Breaker	
Tractor Shovel	
Shotcrete Machine	
Drilling Jumbo	
Dump Truck	
Truck Mixer	

Table 6-3: Standards of Gas Emission for Construction Machineries (g/kw,h) (Reference Value)

Standards	Output: P(kW)	HC	NOx	CO	PM	Black Fumes (%)
Primary Standards	7.5 ≦ P < 15	2.4	12.4	5.7	-	50
	15 ≦ P < 30	1.9	10.5	5.7	-	50
	30 ≦ P < 272	1.3	9.2	5.0	-	50
Secondary Standards	8 ≦ P < 19	1.5	9.0	5.0	0.8	40
	19 ≦ P < 37	1.5	8.0	5.0	0.8	40
	37 ≦ P < 75	1.3	7.0	5.0	0.4	40
	75 ≦ P < 130	1.0	6.0	5.0	0.3	40
	130 ≦ P < 560	1.0	6.0	3.5	0.2	40
Tertiary Standards	8 ≦ P < 19	7.5		5.0	0.8	40
	19 ≦ P < 37	1.0	6.0	5.0	0.4	40
	37 ≦ P < 56	0.7	4.0	5.0	0.3	35
	56 ≦ P < 75	0.7	4.0	5.0	0.25	30
	75 ≦ P < 130	0.4	3.6	5.0	0.2	25
	130 ≦ P < 560	0.4	3.6	3.5	0.17	25

6.3.2 Natural Gases

The additional attention shall be given in case the construction section is located in the area of gas, oil, coalfield, or hot spring with geologically fractured zone, mudstone or coal layer, etc. as the flammable gas, noxious gas, and other natural gases likely to spring out. If the explosion of the flammable gas is expected, the wind speed must be at least 0.5m/s to prevent the backflow of methane. Furthermore, the wind speed must of 1m/s or more is recommendable to dissipate methane rare gas if the methane is constantly springing out. The ventilation, prohibition of the use of fire, use of explosion protection on the equipment inside the tunnel, evacuation, and prohibition of the entry into the

tunnel must be considered depending on the measurement of the gas detector. Table 6-4 shows the allowable amount of natural noxious and flammable gas. The legal limits in the table are determined based on the presence or absence of danger. On the other hand, the ACGIH (American Conference of Governmental Industries Hygienists) values is in which it will not affect the worker even if they are working continuously inside the tunnel. The values of ACGIH are used for the design of ventilation facilities. In a poorly ventilated area, workers may become hypoxic due to the consumption of air oxygen or the generation of oxygen-deficient air. The main items regulated in the Industrial Safety and Health Act in Japan on the natural noxious gas are as follows;

1) Carbon dioxide concentration standard in the tunnel
   The carbon dioxide concentration in the workplace in the tunnel shall be less than 1.5%. However, this does not apply to the work related to rescue or injury prevention using air respiratory or hose mask.

2) Prohibition of trespassing
   No one other than the authorized personnel is allowed to enter the area where the carbon dioxide concentration exceeds 1.5%, the oxygen concentration is less than 18%, or the hydrogen sulphide concentration exceeds 10ppm.

Additional attention is necessary to the tunnels in the sandy gravel layer, where the pneumatic method is used nearby, or in the survey pits, vertical and inclined shafts where ordinary ventilation is difficult. In order to prevent health problems caused by high temperature and humidity, appropriate measures such as ventilation and air conditioning must be undertaken to control the temperature and humidity.

Table 6-4: Noxious Gas and Flammable Gas in Tunnel (Reference Value)

Gas	Color, Odor etc.	Expected intoxication, disability etc.	Specific Gravity (Air1.0)	Explosion Range (Vol %)	Legal Restriction (Japan)	Permissible Concentration (ppm)	
						Japan Society for Occupational Health	ACGIH
CO	Colorless, Odorless	Intoxication etc.	0.97	12.5 ~ 74.0	Below 100 ppm	50	25
CO_2	Colorless, Odorless	Oxygen insufficiency, Intoxication	1.53	-	Below 1.5 %	5000	5000
NO	Colorless, Irritant odor	Intoxication	1.04	-	-	-	25
NO_2	Reddish-brown, Bluish-yellow, Powder smoke odor	Intoxication	1.59	-	-	Under consideration	0.2
SO_2	Colorless, Sulfurous odor	Intoxication	2.26	-	-	Under consideration	0.25 (max)
H_2S	Colorless, Putrid odor	Intoxication	1.199	-	Below 1 ppm	5	1
HCl	Colorless, Irritant odor	Intoxication	1.27	-	-	2 (max.)	2 (max.)
Anoxic Air (O_2)	Colorless, Odorless	Anoxic	1.11	-	Above 18 %	-	-
Excess Air (O_2)	Colorless, Odorless	Intense combustion	1.11	-	-	-	-

HCHO	Colorless, Irritant odor	Intoxication	1.07	-	Below 0.1 ppm	0.1	0.3 (max)
CH_4	Colorless, Odorless	Explosion	0.55	5.0 ~ 15.0	Below 1.5 %	-	-
C_2H_2	Colorless, Etheric odor	Explosion	0.91	2.5 ~ 100	-	-	-
C_3H_8	Colorless, Odorless	Explosion	1.56	2.2 ~ 9.5	-	-	-
NH_3	Colorless, Irritant odor	Intoxication , Explosion	0.597	15.0 ~ 25.0	-	25	25

6.4 Passage

In order to prevent accidents caused by vehicles (rail-type, tyre-type, etc), it is necessary to provide safe passages for workers. The passage shall have enough space to avoid contact among workers and vehicles. The surface of the passage shall be maintained and appropriately illuminated so the workers can walk safely at all times. In addition, it is preferable that the passages and vehicle paths are clearly distinguished by safety ropes or anything similar.

If there is rail-type transportation in the tunnel, the distance between the rail vehicle and the sidewall or obstacle must be 0.6m or more on one side. In case the mentioned distance cannot be acquired (small section of the tunnel), measures to keep workers away from the rail vehicle shall be undertaken by providing clear shelter at appropriate intervals, installing traffic signals, assigning monitoring personnel, etc (mentioned in the ordinance on Industrial Safety and Health Act of Japan). In the case of tyre-type vehicles, it is preferable that the distance between the vehicle and the sidewall to be more than 1m (excluding vehicles passing each other).

Temporary passages should be durable with the slope less than 30 degrees (excluding passage with stair or durable handrail with a height less than 2m). A footstep bar(step) or non-slip grip shall be installed for slopes greater than 15 degrees. In addition, in the case of the section where there is a danger of falling, handrails with a height of 85cm or more and a middle rail with a height of between 35cm to 50cm shall be installed. The landing space shall be installed every 10 or less if the length of the temporary passages is more than 15m and every 7m or less for the climbing bridge higher than 8m.

Figure 6-1: Example of Temporary Passage during Tunnel Construction[1]

[1] Ministry of Health, Labour and Welfare

For ladder passages, the slope must be within 80 degrees. If the distance of the ladder passage is more than 10m, a landing shall be installed every 5m or less and the upper ladder protrudes more than 60cm from the floor. In the vertical and inclined tunnel, a partition wall or other separator shall be installed to avoid contact between hoisting equipment and workers.

6.5 Safety Inspection

During construction, deformation and defect may occur in the earth, tunnel supports, working environment, machinery, and equipment which may lead to disasters. Therefore, safety inspections shall be undertaken appropriately during construction to prevent disasters caused by these deformations and defects.

It is important to add the contents of the safety inspection conforming to the actual site situation considering the construction status, machinery, and equipment to be used in addition to the contents stipulated by the law (if any). Particularly necessary items shall be inspected by the person in charge or a designated inspector.

The main items of inspections are as follows;

1) Earth: Loose rocks or cracks in the tunnel face, deformation in the uncovered section, flammable or noxious gas, deformation on the surface of the spring.
2) Tunnel Supports cracks or detachment on the shotcrete, rock bolt fixing, plate deformation, bolt breakage, settlement and deformation of steel support, etc.
3) Work Environment: temperature, humidity, wind speed, atmospheric pressure, oxygen concentration, visibility, ventilation volume, exhaust gas, dust concentration, vibration, and noise, etc.
4) Machines and Facilities: maintenance status of passages, access roads, tracks, vehicles, ventilation equipment, lighting equipment, drainage, communication and notification equipment, emergency evacuation equipment and rescue equipment, etc.

In order to carry out effective inspections, it is preferable to consider the inspection period for each inspection item and prepare a standard inspection list (checklist). The result of the inspection shall be recorded and stored if necessary. The example of the checklist is shown in Chapter 5 (Figure 5-11: Example of face mapping report in road tunnel).

If any abnormalities are found from the result of the inspection, repairs and other appropriate measures shall be taken immediately.

6.6 Labour Health

In a confined spaces such as tunnels, harmful substances such as dust are difficult to be diluted and noise attenuation is small. There is also a risk of health due to the handling of vibrating tools. Therefore, measures to prevent the risk of the health of workers must be undertaken such as improvement of machinery, work methods to maintain a good work environment. In order to ascertain the health status of workers, regular health examinations and health education shall be considered.

The table below shows the standards for the working environment evaluation indicator and the measurement tools etc.

Table 6-5: Environmental Evaluation Checklist (Reference Value)[2]

Check items	Place	Measuring location	Frequency	Measuring tools
Dust concentration (2.5 mg/m^3)	50m from the tunnel face and beyond	50m+ from the tunnel face	Once in half a month	Digital dust indicator
Wind velocity in tunnel (0.3 m/s)	Inside pit	50m+ from the tunnel face	Once in half a month	Hotwire anemometer
Temperature in tunnel (28 degrees Celsius)		Same with dust concentration measurement location	Same with dust concentration measurement	Digital thermometer
CH4 concentration (1.5%)		Location with risks of pollutant generation	Appropriately	Gas measurement instrument
CO (50 ppm)		High workload location	High workload duration	
NOx (25 ppm)				
Hourly SPM (0.2 mg/m^3)	Outside pit	Pithead	Appropriately	Dust indicator
Supply air temperature				Hotwire anemometer

The health problem and its preventive measures that require additional attention are as follows;

6.6.1 Temperature, Humidity, Wind and Atmospheric Pressure

The temperature, humidity, wind speed, and air pressure in the tunnel are the important factors for ensuring the safety and health of the workers. It is preferable to maintain the temperature below 28 degrees Celsius (temperature which does not cause discomfort) and wind speed of about 0.3m/s (if there is no flammable gas). Also, the additional measure is necessary for winter as concentration and workability may decrease in the cold environment.

6.6.2 Dust

A large amount of dust is generated during blasting, drilling, stacking, waste dumping, etc, and inhaling this dust for a long period may cause pneumoconiosis. The target level of the dust concentration (inhalable dust) shall be less than 3mg/m^3[3] at the point of 50m from the tunnel face. In case the dust concentration could be reduced to less than 3mg/m^3 at the smaller section of the tunnel, use the lowest value possible. The use of respiratory protective equipment with a motor fan is recommendable for work in a dusty area. Dust concentration in the tunnelling work varies depending on many factors such as geological conditions, tunnel cross-section, construction method, machineries, and ventilation system. It is recommendable to measure the dust concentration regularly once every six months or less and adjust the air volume of the ventilation system according to the result[4].

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[2] Yoshitsugu Kobayashi, Toru Tnaka (2016); Development of the Tunnel Gallery Environment Unitary Management Automatic Control System, Toda Corporation Technical Report42

[3] Guidelines of Ministry of Health, Labour and Welfare (Japan)

[4] Ordinance on Prevention of Hazards Due to Dust, Ministry of Labour Ordinance No.18 of April 1979

Generally, the dust shall be prevented from the source. However, if the dust cannot be prevented at the source, the dust collector may be effective to prevent the diffusion of the dust. In addition, dilution by the fresh air through ventilation is required.

6.6.3 Noise

Noise not only causes discomfort, but also interferes with signals by voice or sound, and adversely affects physiological functions. Furthermore, long exposure to noise can cause noise-induced hearing loss. The degree of hearing loss increase as the volume and the period increase. If the volume of the sound is the same, the high-frequency components are harmful than the low-frequency components. It is advisable for the person in charge to measure the noise at the location where the drilling and stacking work is undertaken and necessary measures as mentioned below shall be taken.

1) Noise more than 85db(A) and less than 90db(B)
   Use of noise protection for the workers engaging directly with the noise work if necessary.

2) Noise more than 90dB(A)
   Ensure the use of noise protection and signboard of the use of noise protection is visible to the workers.

In the case of Japan, the employer has to submit the documents or drawing regarding the noise prevention measures which includes [work area of bolt removing and tightening, nut runner, impact wrench], [work area for rock crushing], [work area involving drilling machinery and stacking], etc.

Table 6-6 shows the acceptable values of noise protection by the Japan Society of Occupational Health. In order to prevent noise interference, it is essential to select equipment, machinery, and work method with less noise as well as install soundproof devices to equipment and machinery to reduce noise. If there is a risk of noise-induced hearing loss due to long exposure to the noise, ear protection and other protective equipment shall be used appropriately.

Table 6-6: Permissible Noise Standards for Hearing Protection (Reference Value)

Center Frequency (Hz)	Permissible Octave Band Level (dB)*					
	480 min	240 min	120 min	60 min	40 min	30 min
250	98	102	108	117	120	120
500	92	95	99	105	112	117
1000	86	88	91	95	99	103
2000	83	84	85	88	90	92
3000	82	83	84	86	88	90
4000	82	83	85	87	89	91
8000	87	89	92	97	101	105

* Even in case audio disclosure per day is 8 hours, continued over 10 years, hearing loss is expected to be below 10 dB when the audio frequency is below 1 kHz, below 15 dB when 2 kHz, 20 dB when above 3 kHz.

6.6.4 Vibration

Machineries and tools with vibration such as jackhammers are commonly used in the tunnelling work. These vibrations can cause vibration disorders such as the Raynaud phenomenon, peripheral neuropathy, bone and joint deformation to the hands and arms of workers.

As a measure to prevent vibration disorder, it is necessary to select machinery or tool that generates as little vibration as possible or has vibration isolation. Also, the work plan shall be properly managed by considering the engaging time in vibration work. The medical examination in vibration disorders shall be undertaken during hiring, reassignment, and every six months thereafter.

6.7 Prevention of Fire and Explosion

6.7.1 Prevention of Fire

If a fire occurs in the tunnel, there is a risk of causing serious disaster due to the following reasons;

1) Workers are trapped at the far end than the point where the fire occurred.
2) Carbon monoxide poisoning as the tunnel is filled with noxious gases such as fumes and carbon monoxide generated by fire.
3) Secondary disasters may be caused due to panic in the event of a power outage.
4) The collapse of the wall or tunnel supports due to damage or deformation caused by fire.

The basis of fire prevention measures is to manage the fire sources and combustible materials to eliminate the danger of fire ignition. Therefore, it is important to designate a person in charge to monitor the use of flammable items. After the use of flammable items, it is necessary to prevent the occurrence of fire due to embers. In addition, it is important to keep the tunnel clean and minimize flammable materials.

However, a prompt initial fire extinguish is the most important point in the event of a fire. The suitable fire extinguisher equipment shall be installed and its locations shall be informed to the workers. The fire drills and the use of the equipment shall be conducted regularly. The fire extinguisher equipment shall be constantly inspected to maintain its functions.

Fire-extinguishing equipment including fire-extinguishers must be appropriate for the condition of the tunnels and the expected ignition source. Maintenance and inspection need to be conducted regularly and be recorded.

Fire-extinguishing equipment in the tunnel should be selected with the following points;

1. Functions properly even in facilities with high humidity.
2. Small and high fire extinguishing capacity unit. It should not cause invisibility when using it.
3. The same model of extinguishers should be allocated in the same tunnel in order to avoid misoperation.

Generally, ABC powder extinguishers or reinforced liquid extinguishers are used in tunnel construction. Other types of extinguishers are mentioned in the table below.

Table 6-7: Types of Fire Extinguisher[5]

Types	Details	Dominant component	Applicable types of fire		
			Normal	Oil	Electric
Powder type	Powder (ABC) fire-extinguishers	Ammonium dihydrogen phosphate	●	●	●
	Powder (Na) fire-extinguishers	Sodium hydrogen carbonate		●	●
	Powder (K) fire-extinguishers	Potassium bicarbonate		●	●
Liquid or bubble type	Liquid (wetting out agent etc.) fire-extinguishers	Pure water, wetting out agent	●		●
	Reinforced liquid fire-extinguishers	Potassium carbonate	●	●	●
	Neutral reinforced liquid fire-extinguishers	Fluorinated surfactant	●	●	●
	Mechanical foam (Aqueous film forming foam concentrates) fire-extinguishers	Fluorinated surfactant	●	●	
	Chemical foam fire-extinguishers	Agent A sodium bicarbonate Agent B aluminum sulfate	●	●	
Gas type	Carbon dioxide fire-extinguishers	Carbon dioxide		●	●
	Halon 1301 fire-extinguishers	Halon 1301		●	●

Applicable types of fire
Normal fire: fires caused by burning woods, papers, and fibers, etc.
Oil fire: fires caused by flammable liquid (petroleum etc.), fats and oils, etc.
Electric fire: fires caused by electric equipment and electric appliance etc.

6.7.2 Prevention of Explosion

If there is a flammable gas inside the tunnel or the possibility of having one, assess the type, properties, presence status, generation status, and remove anything that could become a fire source before conducting sufficient ventilation to dilute to a safe concentration range. If the dilution to a safe concentration range is difficult only by increasing the ventilation equipment, gas venting through boring will be necessary to release the direct gas outside of the tunnel or to a non-noxious location.

There are a wide variety of fire sources that may cause explosions due to flammable gas in the tunnel such as open flames, ignition devices, smoking equipment, flames for gas fusing, electric sparks, blasting flames, friction and impact sparks, etc. Appropriate measures must be taken against these potential sources of fire such as the prohibition of bringing into the tunnel, proper work methods, reliable grounding to avoid electrification the use of explosion-proof equipment.

During the construction, the concentration of flammable gas may reach the explosion limit partially (upper and lower part of tunnel face). Therefore, it is necessary to circulate the gas around the tunnel face with the air outside to reduce the gas concentration of flammable gas. For example, if the tunnel has an upward slope toward the tunnel face, the gas detector shall be installed at the upper part of the face as the methane gas has lower specific gravity than air and the wind speed shall be set so that the reading value

[5] Ministry of Agriculture, Forestry and Fisheries; Standard of Civil Works Control 2019

of the gas detector is below the reference value at all times. On the other hand, the gas detector shall be installed at the lower part since hydrogen sulphide has higher specific gravity than air. In addition, it is recommended to install a gas detector during the lining concrete where there is an obstruction of airflow.

In case the appropriate installation of a gas detector is difficult, the gas detector shall be installed downstream from the location of the gas spring considering the ventilation direction.

The following points shall be given attention after a long-term suspension of work.

1) Measuring the concentration of flammable gas and oxygen in the tunnel and confirming that safety has been ensured before entering the tunnel.
2) Starting the ventilation system from a safe place outside of the tunnel.

6.8 Emergency Measures

6.8.1 Emergency Response

The evacuation of the workers to a safe place must be prioritized during an emergency such as the collapse of the tunnel face, abnormal flooding or fire, occurrence of noxious gas, etc.

Contractors must define the corresponding policy for emergency operation (ex. accidents etc.).

1) Lifesaving (highest priority)
2) Establish an emergency contact/communication system
3) Response procedure in case of emergency
4) First aid measure
5) Reporting accidents and injuries

6.8.2 Warning, Evacuation, and Rescue Aid

(1) Warning

The prompt and accurate transmission of the information is essential during the emergency situation to avoid further disaster. Therefore, it is necessary to have appropriate emergency warning equipment at the construction area and establish a system that enables prompt communication with related people.

(2) Evacuation

The emergency passages shall be kept clear from obstacles and emergency lights shall be installed in appropriate intervals. Equipment related to the emergency evacuation shall be placed appropriately inside the tunnel. In addition, it is necessary to provide the workers with knowledge related to the education by conduction safety education and evacuation drills. Also, installation of communication (wired or wireless) inside the tunnels is preferable in order to communicate with the workers outside the tunnels

In case workers find any abnormal incidents during the works, they need to report to the person in charge immediately and conduct the emergency operation according to the orders.

1) Any indications at the tunnel face of occurring cave-in or collapse because of spring, rockfall and rumbling, etc.
2) A sudden change of spring amount or its turbidity

3) Any abnormalities (ex. noise, floating, bending, breaking) are observed
4) Cracks in the concrete coatings
5) Any changes in the density of hazardous gas etc.
6) Others, if necessary in regard to safety management

(3) Rescue Aid

If there are survivors inside the tunnel during the emergency occurrence, rescue measures shall be taken immediately based on the comprehensive judgement among the related parties. In the event of an explosion or fire in the tunnel, rescue activities shall be undertaken by trained first-aid personnel (shall be selected for a tunnelling work with a distance of more than 1km or a depth of 50m or more) and equipped with appropriate tools for rescue[6]. During the rescue, the employer shall take necessary preparatory measures in advance to prevent further damages caused by the secondary disasters[7]. In addition, air respiratory equipment, oximeter, portable lighting, and other necessary tools for rescue shall be well maintained at all times.

Reference Value is based on "Japan Society of Civil Engineers (2006); Standard Specifications for Tunneling - 2006"

[6] Industrial Safety and Health Act (Japan)
[7] Notification of Labor Standards Bureau (Japan)

CHAPTER 7 Tunnel Ventilation

7.1 Requirement

The objective of ventilation in the road tunnel is to secure safe and comfortable against the passage of tunnel users and to secure the environment for various management work in the tunnel. Specifically, tunnel users and maintenance workers are not physiologically affected by harmful substances in the tunnel, and the visibility inside the tunnel does not deteriorate. Therefore, ventilation to control air pollution with the exhaust fumes of the car is necessary for the road tunnel. And, the required ventilation volume of the tunnel shall be calculated and must be considered ventilation method. And if there is a change in the tunnel structure, the road traffic plan, etc., the required ventilation volume and the ventilation method are required to be reexamined each time.

7.2 Criteria

The ventilation has natural ventilation and mechanical ventilation. The natural ventilation volume of the tunnel can be calculated by the following factors; the traffic, a traffic method (two-way traffic, one way), tunnel structure (cross-section, longitudinal slope, tunnel alignment), environmental (highlands, low mountainous district, low land, the shore, city, residential area) and meteorological phenomena (the wind pressure, the direction of the wind, the rain, temperature, the humidity).
In case of less than the required ventilation volume of the tunnel, the tunnel cannot provide a safe and smooth running environment to users due to become insufficient ventilation. Therefore, machine ventilation should be installed to eliminate this ventilation shortage. In principle, the installation of tunnel ventilation follows the above concept. Moreover, the criteria for installing ventilation in the tunnel shall be in accordance with the guidelines indicated in 'IRC:SP: 91-2010 GUIDE LINES FOR ROAD TUNNELS (*1)', and the special mention for the tunnel ventilation on mountain roads is mainly noted in this chapter.
(*1) IRC: The INDIAN ROADS CONGRESS

7.3 Natural ventilation

Natural ventilation is ventilation using the capability of nature, and there are wind-forced ventilation and gravity ventilation.
Wind-forced ventilation is a situation where the wind velocity occurred because of the difference in the wind pressure between tunnel portals. In addition, this wind velocity is also changed by the vehicle traveling speed. Generally, this ventilation volume is proportional to the wind velocity of the tunnel. The gravity ventilation is the wind velocity occurring because of specific gravity differences and atmosphere to occur because of the difference of temperature inside and outside the tunnel (buoyancy). Generally, this ventilation volume is inversely proportional to the position (position of aeration and the exhaust) of the vent, and inversely proportional to the difference in temperature, and proportional to the cross-section of the portal. In any above case, it is difficult to quantitatively determine the amount of ventilation of natural ventilation (natural ventilation capacity) because of unstable natural conditions.

Therefore, Natural ventilation generally shall be applicable for tunnels of length up to 500 [m] as mentioned in 'IRC:SP: 91-2010'. However, for tunnels of length, more than 250 [m] natural ventilation systems shall be used only after a thorough evaluation of reliance on natural ventilation especially with reference to effects of meteorological and operating conditions. Moreover, if the general discharge volume of exhaust fumes can be reduced, this length can be extended as well.

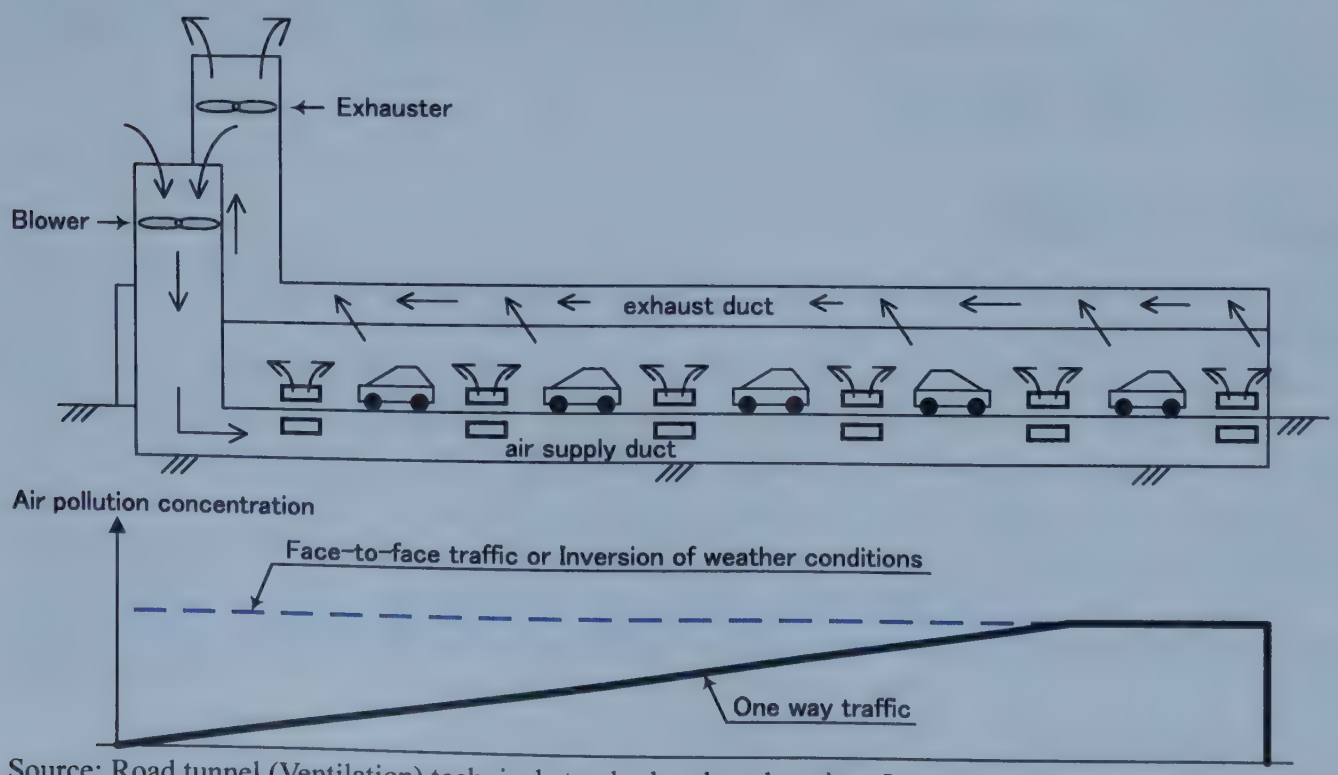
7.4 Mechanical ventilation

Mechanical ventilation basically shall be provided in case of tunnels of length more than 500 m according to above mention.
Mechanical ventilation of tunnels on mountain roads is consisted of the following three types of ventilation systems used separately or in combination. Moreover, the linear ventilation system is generally less effective than the transverse ventilation system in terms of ventilation, but economically it's more viable.

- Transverse ventilation system
- Semi transverse ventilation system
- Linear ventilation system

7.4.1 Transverse ventilation system

In the Transverse ventilation system, an air supply duct and an exhaust duct are arranged outside the roadway in the tunnel to generate airflow as air supply and exhaust transverse along the roadway so that these airflows ventilate the tunnel. As compared to others this system is unlikely to be affected by weather conditions and face-to-face traffic, and it is possible to uniformly distribute the air distribution and the exhaust fumes quickly, and it is possible to keep the air supply pressure constant inside the tunnel.
Construction cost is highest because air supply duct and exhaust duct are necessary. The air pollution concentration in the tunnel decreases in proportion to the distance from the exhaust fan.
This system is mainly adopted in a relatively long tunnel (2 to 4 [Km] or more) and a city tunnel where traffic congestion is likely to occur.
Figure 7-1 shows the configuration of a typical transverse ventilation system and air pollution concentration graph.

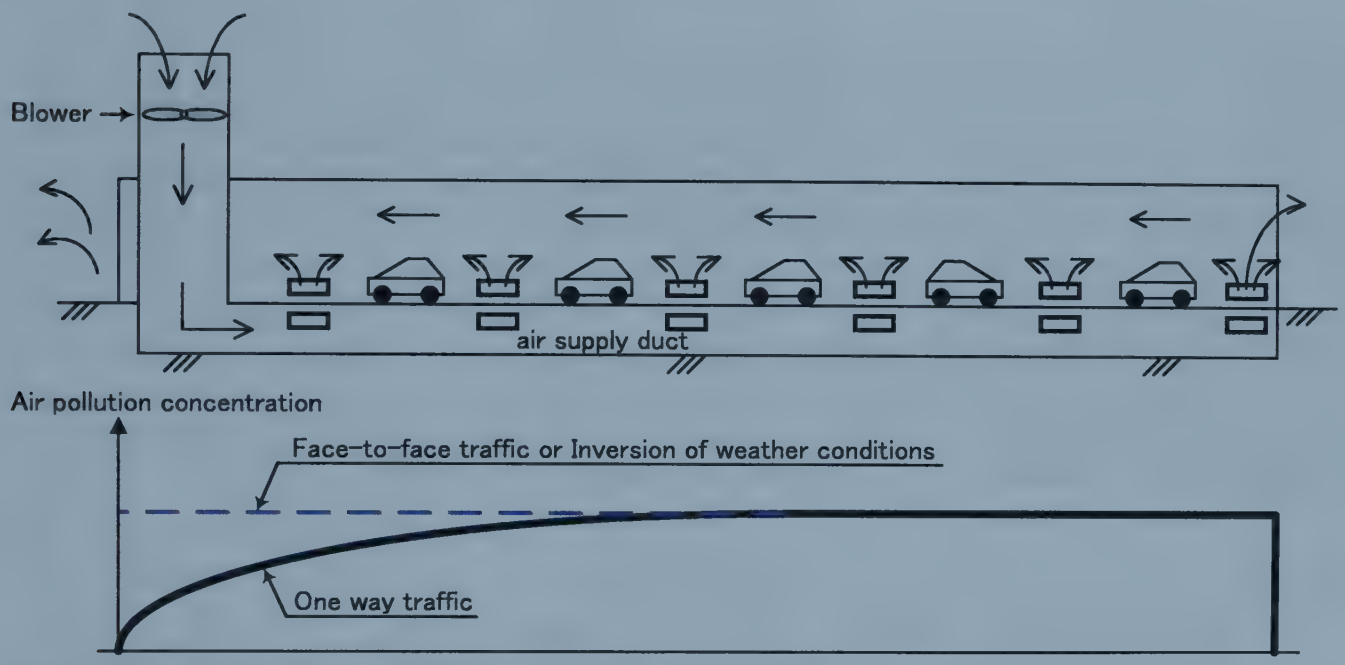


Source: Road tunnel (Ventilation) technical standard and explanation; Japan Road Association 2010.

Figure 7-1: Transverse ventilation system

7.4.2 Semi Transverse ventilation system

In the Semi Transverse ventilation system, an air supply duct or an exhaust duct is arranged outside the roadway in the tunnel to generate airflow in the transverse along with the roadway so that this airflow ventilates the tunnel. As compared to others this system is unlikely to be affected by weather conditions and face-to-face traffic.
Construction costs are higher, as air supply ducts or exhaust ducts are required. Air pollution concentration in the tunnel is almost uniform except near the entrance in case of one-way traffic and is uniform throughout the tunnel in the case of face-to-face traffic.
This system is mainly adopted in the medium-sized tunnel (1.5 to 3 [Km]) because the air pollution concentration can be made relatively uniform. Also, when providing a reversible blower for air supply, it is effective even in case of fire.
Figure 7-2 shows a typical semi-transverse ventilation system configuration and air pollution concentration graph.
This is a configuration example in which an air supply duct is installed, and is ventilated by air supply.

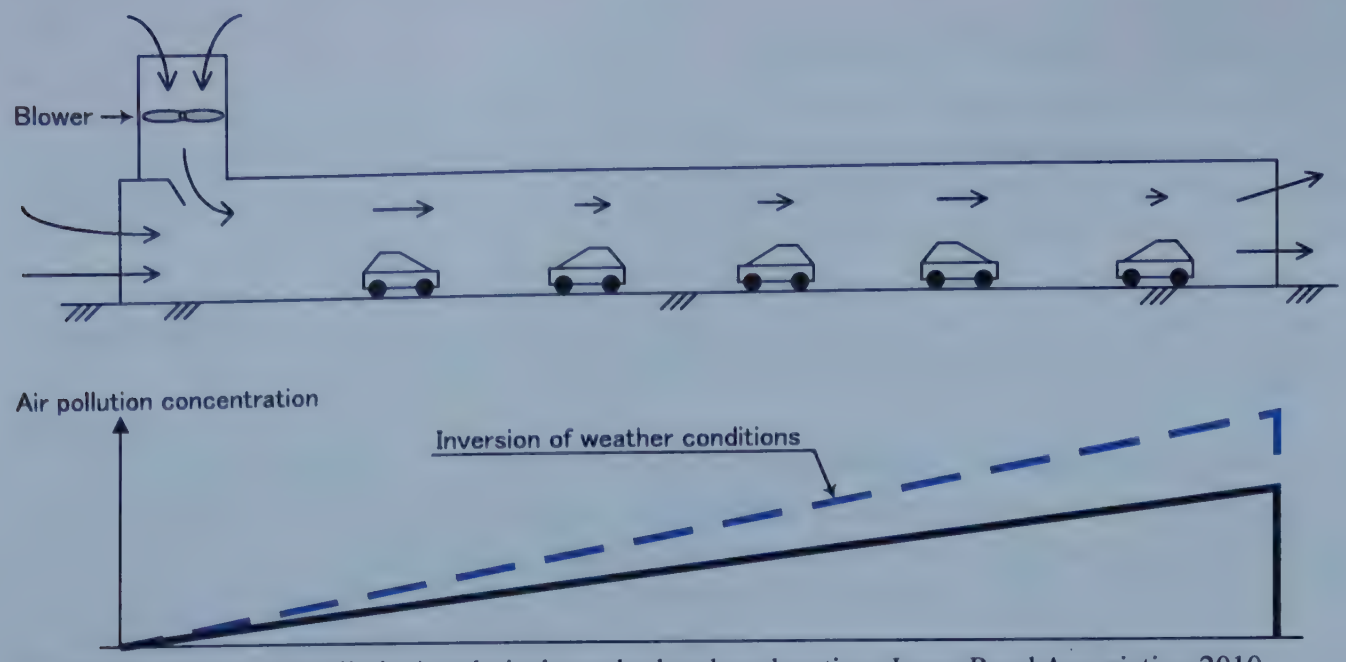


Source: Road tunnel (Ventilation) technical standard and explanation; Japan Road Association 2010.

Figure 7-2: Semi Transverse ventilation system (Air supply)

7.4.3 Linear ventilation system

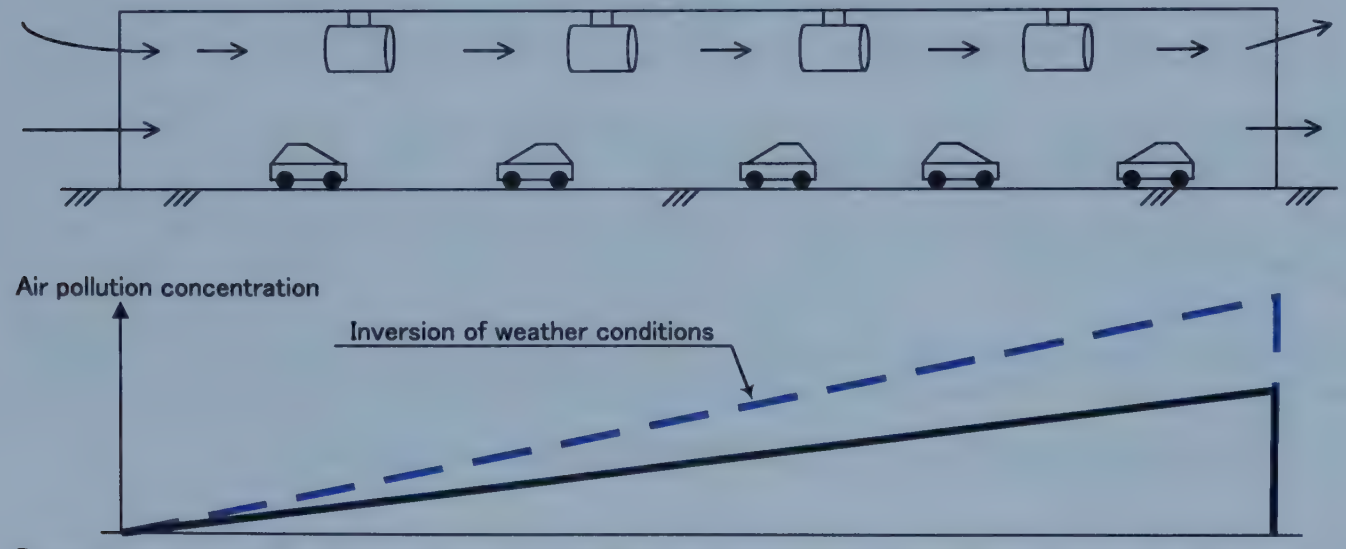
In the linear ventilation system, a blowing or exhausting from a limited place in the tunnel is arranged to generate airflow in linear along with the roadway so that this airflow ventilates the tunnel. In this system, as a requirement of more air flow speed in the tunnel and diffusion of smoke are necessary for ventilation as the tunnel becomes longer. Therefore, the calculation for the ventilation will be complicated and delicate.
Construction cost is the cheapest because air supply ducts and exhaust ducts are not required. Since the ventilation capacity is lower than that of the semi-transverse ventilation system or transverse ventilation system, it is mainly adopted in a relatively short tunnel (1 to 2 [Km]). However, in case of the possibility to arrange the shafts on one-way road at appropriate intervals, it can be adopted by a long tunnel.
Figure 7-3 and Figure 7-4 show a typical linear ventilation system configuration and air pollution concentration graph.



Source: Road tunnel (Ventilation) technical standard and explanation; Japan Road Association 2010.

Figure 7-3: Linear ventilation system (Blower type)

In Figure 7-3; the airflow sent from the fan arranged at the tunnel's end and the other airflow due to the piston action of the automobile become a mixed airflow to generate one airflow in linear along with the roadway so that this airflow ventilates the tunnel. This is effective for one-way traffic, and the wind speed in the tunnel is constant. Air pollution concentration in the tunnel is highest at the exit.



Source: Road tunnel (Ventilation) technical standard and explanation; Japan Road Association 2010.

Figure 7-4: Linear ventilation system (Jet fan type)

In Figure 7-4; the jet fans at appropriate intervals are arranged to hang the tunnel's ceiling panel to generate an airflow in linear along with the roadway so that this airflow ventilates the tunnel. The installation space in the electrical machine room for the control equipment of the blowing fan and exhaust fan won't be required. However, the tunnel ceiling needs to be slightly higher. Air pollution concentration in the tunnel varies depending on the direction of airflow by the jet fan. At the end of the tunnel the wind exhaust side is the highest.

7.5 Plan

Tunnel ventilation is related to the following factors;

- The structure of the main body of the tunnel,
- The basic plan of tunnel and
- The closeness includes the route and alignment.

Ventilation Plan is not only required to choose the most economical machine ventilation but is also required to examine the influence to give on a traffic method, a disaster prevention plan (non-common use facilities), and the neighboring environment. In addition, as for the ventilation plan, a future ventilation scale should be considered in advance because there will be extreme difficulties in repairing the main body of the tunnel.

(1) Planning procedure

The general flow of the plan, investigation, design, and construction is as follows.

1) Route planning of tunnel
2) Collection of material necessary for the design of ventilation facilities
3) Outline design of amount of ventilation and ventilation system
4) Calculation of the amount of ventilation
5) Calculation of ventilation pressure and Decision of ventilation system
6) Design of ventilator
7) Production and construction of ventilator
8) Beginning of operation

(2) Points of concern

1) Relation to the main body of the tunnel

The longitudinal slope greatly influences the designed amount of ventilation. Moreover, the cross-sectional shape and lining of the tunnel greatly influence the position of the shaft for ventilation, the inclined passage, electric dust collection machine room for ventilation where the intake, trunk air outlet, and the ventilator (Jet fan, etc.) are set up. Therefore, appropriate ventilation with feasibility should be confirmed by the outline ventilation calculation during the route planning phase of the tunnel.

2) Influence on surroundings

The ventilation plan should be considered for environmental protection because of "Exhaust diffusion from the tunnel pithead and the ventilating tower", "Noise and vibration from the ventilator", and "Spectacle in the ventilating tower".

3) Consideration on disaster prevention

The ventilation facilities have a disaster prevention function that pushes out smoke outside the tunnel compulsorily. As a result, in case of a fire in the tunnel, the shelter environment can be secured.

7.6 Investigation

The ventilation plan will carry out traffic investigation, meteorological surveys, environmental research, geographical features, and geological surveys as necessary.

(1) Traffic investigation

Traffic investigation shall be conducted to obtain to be used in calculations of predetermined ventilation amount and predetermined environmental influences. Depending on this information and the structure of the tunnel, the required ventilation volume would be calculated. This investigation is not limited to the total number of vehicles, but also considers the vehicle type composition, distribution by direction, time, day of the week, seasonal variation, and large car traffic volume. It has the amount of exhaust and the traffic ventilation power of smoke and the carbon monoxide of the car as a calculation element necessary. Factors necessary for the calculation of ventilation

volume and ventilation calculation are car smoke emissions, carbon monoxide emissions, and traffic ventilation capability as well. This amount of exhaust changes depending on the kind and the size of the vehicle motor. The traffic ventilation capability changes depending on the size and the model of the vehicle body. Therefore, the number of the car engine organization (diesel and fumes line) and the displacement and each model is necessary for data concerning traffic as well.

(2) Meteorological surveys

Meteorological surveys shall be conducted to obtain the necessary basic data for estimation of natural ventilation, estimation of the influence of weather on mechanical ventilation, and prediction of the influence of air discharged from the tunnel to the surrounding environment. Meteorological surveys are carried out on atmospheric pressure differences between the portals, wind direction, wind speed, temperature, and humidity, but it is also important to change observation items and methods according to research purposes.

(3) Environmental research

Environmental research shall be conducted to formulate a ventilation plan that takes into consideration the characteristics of the routes and the road conditions around the tunnel. If there is a residential area near the tunnel, measures and surveys of the noise and various fumes concentrations should be considered in order to evaluate the influence of the tunnel on them.

(4) Geographical features and geological survey

Geographical features and geological survey shall be conducted to obtain the basic data necessary for ventilation shafts, inclined shafts, structures of ventilation facilities, and the construction plan for approach roads.

7.7 Traffic volume

In the case of using traffic volume in ventilation design, the traffic volume should use the Design traffic volume and the Design traffic capacity (N_D [units/hour]) of the road including the tunnel in principle. It is necessary to decide by considering the future traffic volume forecast and economic investment effect so that the size of the ventilation facility does not change easily after operation.

7.8 Harmful substances

7.8.1 Harmful substances

(1) Exhaust fumes

All vehicle type generates obnoxious exhaust fumes and soot with different characteristics as follows. The physiological effects of these harmful substances are also described in 'IRC:SP: 91-2010'.

- Carbon monoxide [CO]
- Carbon Dioxide [CO_2]
- Nitric Oxide [NO] and Nitrogen Oxide [NO_2]
- Sulfur Dioxide [SO_2]
- Hydrocarbons

Ventilation makes the concentration of carbon monoxide dilute to a safe level and also makes the other harmful substances dilute to a safe level in general.

Therefore, from the viewpoint of cost and efficiency, only the concentration of carbon monoxide is managed without controlling the concentration of all harmful substances.

(2) Carbon monoxide

The maximum allowable concentration of carbon monoxide to the human body is 50 ppm on industrial hygiene general, in the case of normal working hours. However, this value is not appropriate. Actually, the work can be carried out during the time zone when the traffic volume is the smallest.
Therefore, the designed concentration of Carbon monoxide is set at 100 [ppm] as the actual staying time in the tunnel is for 30 [minutes] to 1 [hour].

(3) Soot

The visibility inside the tunnel will be obstructed by black smoke emitted from diesel vehicles, earth and sand adhering to the vehicle tire, and wear debris generated by asphalt and tires. These are present in the tunnel as soot.
By PIARC (the Permanent International Association of Road Congresses), the relationship between transmittance and comfort in the tunnel is reported as shown in the table below.

Table 7-1: Relationship for Permeability and Situation in the tunnel

Permeability in tunnel [%]	Situation in tunnel
More than 60	Clear
50 to 60	Be visible
40 to 50	Be annoying
30 to 40	Extremely uncomfortable
Less than 30	Unacceptable

Source: PIARC (World Road Association)

The soot concentration has affected the vehicle to travel safely and comfortably. On the other hand, safe driving in the tunnel is also related to the brightness of tunnel lighting. Even if the transmittance in the tunnel decreases somewhat due to soot, if the tunnel illumination can secure visibility, safe driving of the vehicle can be maintained. Thus, the design value for soot concentration by each design traveling speed should be experimentally determined the relationship between the permeability required for running safety and the brightness of tunnel illumination.
The design value for soot concentration described below table is derived from the above experiment.

Table 7-2: Relationship for Traveling speed and Acceptable soot concentration in tunnel

Traveling speed [Km/hour]	Acceptable soot concentration [%]
More than 80	50
60 to 80	Proportional relationship between 40 and 50
Less than 60	40

Source: Road tunnel (Ventilation) technical standard and explanation; Japan Road Association 2010.

7.8.2 Design concentration of harmful substances

The harmful substances that ventilate out of the tunnel are carbon monoxide and soot in according to the above mentioned. The concentrations of soot and carbon monoxide allowed in the tunnel are shown in the table below as these design concentrations of harmful substances.

Table 7-3: Design concentration of harmful substances

Design traveling speed [Km/hour]	Design concentration of soot [%] (Transmittance in 100m)	Design concentration of Carbon monoxide [ppm]
More than 80	50	100
60 to 80	Proportional relationship between 40 and 50	
Less than 60	40	

Source: Road tunnel (Ventilation) technical standard and explanation; Japan Road Association 2010.

The soot concentration is generally equivalent to the extinction coefficient calculated by the following equation according to Lambert-Beer's law.

$$K_{VI} = -(\frac{1}{L}) \cdot \ln(\frac{\tau}{100})$$

K_{VI} : Soot concentration [%]
τ : Transmittance of Lighting [%]
L : Length for the transmittance of lighting [m]

These design concentrations of harmful substances (K_{VI}, K_{CO}) should be parameters for calculating the required ventilation volume (Q_{rq}) and the unit ventilation amount (q).

7.9 Emission of harmful substance

The amount of emission for harmful components from vehicles traveling through the tunnel greatly varies depending on the type of car and driving conditions. The composition of the exhaust gas of automobiles is significantly different between diesel and petrol vehicles. Therefore, each matter is also different.
Generally, the influence of black smoke on visibility is a matter for diesel vehicles. And petrol cars primarily matter to the influence of carbon monoxide. Therefore, the type of car would be classified by the diesel vehicles and petrol vehicles about the above classification.
However, the composition ratio of diesel vehicles and petrol vehicles practically would be difficult to estimate at present. Therefore, the classification of large vehicles and small vehicles, in general, is used as the statistical data on the automobile instead of the above classification.

7.9.1 Amount of emission for harmful substance

Regarding the amount of emission for harmful substances, as described above, the calculation method differs depending on the evaluation method of the influence of each harmful substance.
In the case of carbon monoxide, considering the concentration to be exposed and its elapsed time should be important for the physiological effect. Thus, the average concentration in the tunnel would be the design reference value.
In the case of soot in the tunnel, in order to ensure visibility, the soot is required to satisfy the necessary conditions even for a short time. Therefore, the amount of s emission for soot would be considered by using the standard deviation, etc.

7.9.2 Investigation and measurement

The amount of emission for harmful substances should be calculated based on investigation and measurements in actual tunnels.
The following table shows the results of measuring and investigating harmful substances generated in tunnels in Japan over several years. This investigation period is from 2000

to 2007. The emission of soot and carbon monoxide in the following table is averaged over this measurement period. The standard deviation of soot emission during this measurement period is also shown in the table. This average and standard deviation of emissions are calculated separately by large vehicles and small vehicles.

Table 7-4: Statistics data of tunnel harmful substance (soot) in Japan

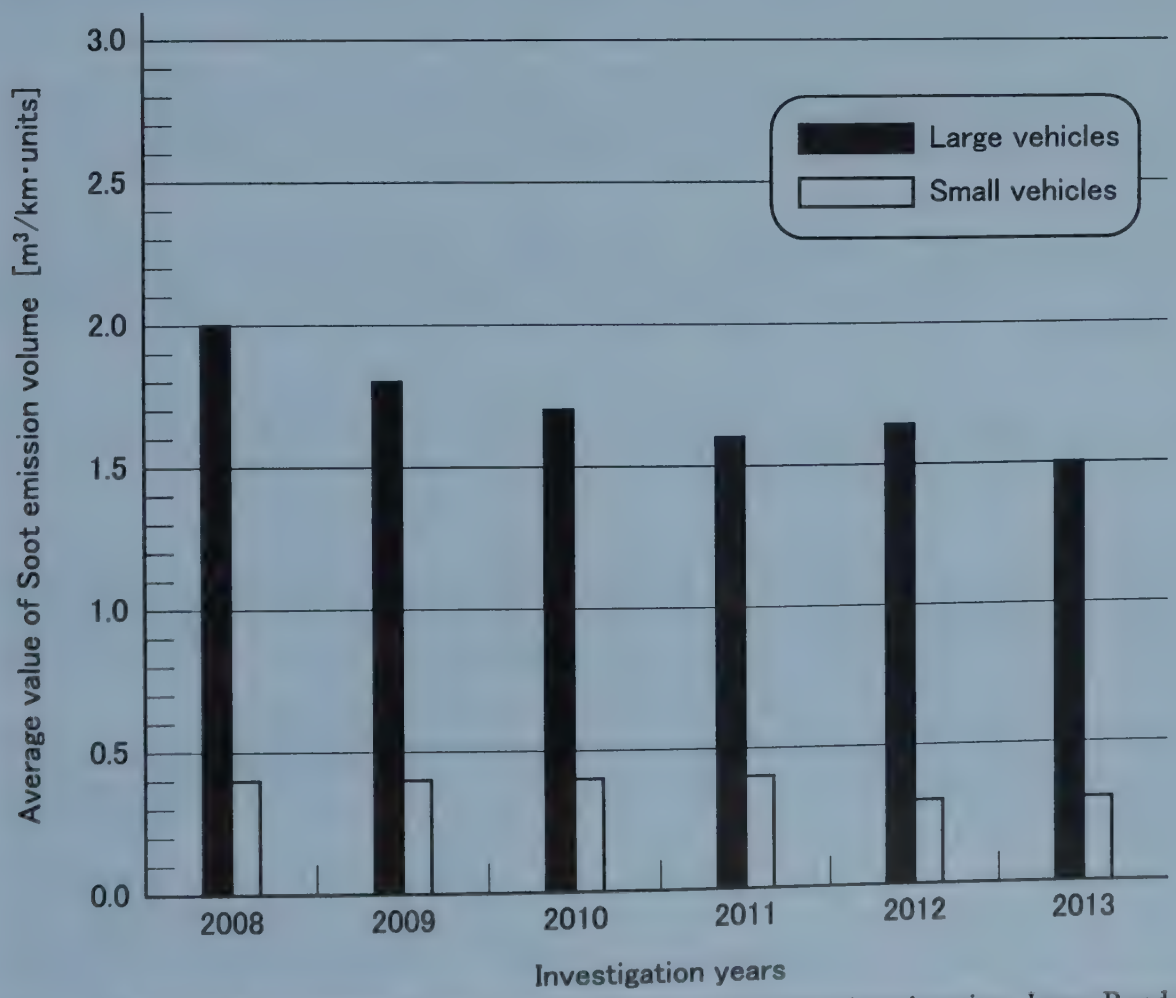
Investigation site	Soot emission from 2000 to 2007 in JAPAN by large vehicles [m^2/km]		by small vehicles [m^2/km]	
	Average	Standard deviation	Average	Standard deviation
A Tunnel	3.195	0.801	0.075	0.031
B Tunnel	3.893	1.103	0.287	0.569
C Tunnel	2.527	0.900	0.607	0.180
D Tunnel	3.941	0.948	0.060	0.351
E Tunnel	1.655	0.691	-	-
F Tunnel	2.263	1.010	0.094	0.076
G Tunnel	1.867	0.588	0.327	0.198

Source: Road tunnel (Ventilation) technical standard and explanation; Japan Road Association 2010.

The amount of emission for soot measured in the above table is the amount of soot generation when one vehicle travels a unit distance (km). Therefore, these amounts of emission for soot in the above table should be a value obtained by multiplying the soot concentration (K_{VI}) by the exhaust amount.

7.9.3 Amount of emission for harmful substance used for ventilation calculation

The emission for soot up to 2007 was scattered data as shown in the table above. These variations gradually disappeared from 2008 to 2013 as shown in the chart below after implementing the exhaust gas regulation in 2005.



Source: Road tunnel (Ventilation) technical standard and explanation; Japan Road Association 2010.

Figure 7-5: L Emissions amount of soot after the exhaust gas regulation in 2005

The typical value for an average and a standard deviation for calculating the amount of emission for soot and carbon monoxide inside the tunnel calculated by the above statics data are shown in the following table. These average and standard deviations in the amount of emission are carried out the consideration on the assumption that the effect of decreasing the soot emission indicated by the replacement of the vehicle to the future electric vehicle and the future strengthened automobile exhaust gas regulation. And, these numbers are when the longitudinal gradient is 0[%], and these are based on actual measurement values in tunnels with a height of 400 [m] or less. Therefore, these should be corrected in case of a gradient road and/or a height 400 [m] or more.
On the other hand, the emission of carbon monoxide also has been confirmed from the above investigation results are found that the amount of emission thereof was decreasing. In the design of the tunnel ventilation facility, the calculation of carbon monoxide emission also shall use the average value in the table below as well as calculation of soot emission. Moreover, in the case of carbon monoxide, even if this concentration rises due to temporary congestion, it will not become a dangerous situation immediately. However, the consideration for this congestion is also required for calculation when the traffic congestion always occurs in the tunnel.

Table 7-5: Average and Standard deviation of emission harmful in Japan

Harmful substances	Vehicle kind	Amount of emission		Remarks
		Average	Standard deviation	
Carbon monoxide	Small	0.005 [m^3/km·units]	Not required	0.007 [m3/km · units] in case of road congestion
	Large			
Soot	Small	0.3 [m^2/km·units]	0.3 [m^2/km·units]	--
	Large	1.5 [m^2/km·units]	0.5 [m^2/km·units]	--

Source: Road tunnel (Ventilation) technical standard and explanation; Japan Road Association 2010.

However, these average and standard deviations in the amount of emission are for use in Japan, and it is required to examine whether it is possible to apply those kinds of values in other places and countries.
Therefore, these numbers must be corrected when future projections are made far investigation on emissions of harmful substances, exhaust gas regulation, vehicle kind replacement will be different.

7.10 Required ventilation volume

7.10.1 General

The require ventilation volume calculates the required ventilation volume of soot and carbon monoxide using the above design concentration of harmful substances (K) as these reference values, respectively. And, the required ventilation volume (Q_{rq}) shall be adopted the largest value calculated from them after this required ventilation.
The amount of soot emission (average value, standard deviation, particulate fraction) from the vehicle and the carbon monoxide emissions (average value, standard deviation) are required to calculate as these required ventilation volumes.
These figures are determined based on surveys and legal automobile emissions regulations. It has also been confirmed by experiments that these emissions not only vary by the type of vehicle but also by longitudinal gradient, traveling speed, and height. Therefore, the emissions of soot and carbon monoxide should be examined for the following items and corrected as necessary.

- Large-scale vehicle contamination ratio
- Correction by traveling speed
- Correction by longitudinal gradient
- Correction by height

The correction coefficient based on the longitudinal gradient and the traveling speed can also be determined by the result of the bench test. However, this test is an effective means only in case that there is the regulation of automobile exhaust fumes. In addition, correction based on height is generally based on the results of the actual driving test and the correction coefficient is generally determined.

7.10.2 Ventilation volume for soot

The design value for soot concentration is defined out as the objective to obtain the necessary viewing environment for the vehicle traveling speed in the tunnel. On the other hand, the emission of soot from vehicles differs depending on the age of the engine, type of the engine, a form of the engine, maintenance status of the engine, load condition of the engine, and driving skill.
Therefore, in the design of the ventilation facility of the tunnel, the displacement of soot is calculated by using a method considering the dispersion of the emissions from each vehicle in general rather than treating the amount of soot emitted by the vehicle as merely an average value. The relationship among the ventilation volume (Q), amount of emission for soot, design concentration (K_{VI} [1/m]) is defined by the equation below.

$$\underbrace{\frac{\mu}{T_t \cdot Q}}_{\text{Average concentration}} + \underbrace{\frac{3\sigma}{\sqrt{2Q \cdot T_t \cdot A_r \cdot 1000}}}_{\text{Dispersion (standard deviation) concentration}} \leqq \underbrace{K_{VI}}_{\text{Specified concentration}}$$

Q : Required ventilation volume per 1 (one) Km of tunnel length [m^3 / (s · Km)]
μ : Average value of soot emission from vehicle group [m^2 / Km]
σ : Standard deviation of soot emission from vehicle group [m^2 / Km]
T_t : Average interval time headway of vehicle group [sec] = 3600 / N
N : Time traffic per tunnel [units/hour]

By the relational formula relating to the above concentration, the equation of the required ventilation volume (Q) is conducted as follows.

$$Q = \left\{ \frac{3\sigma + \sqrt{9\sigma^2 + 8\mu \cdot K_{VI} \cdot A_r \cdot 1000}}{\sqrt{8 \cdot T_t \cdot A_r \cdot 1000} \cdot K_{VI}} \right\}^2$$

The average value (μ) and the standard deviation (σ) of the amount of soot for emission are obtained by the following formula as the mixing ratio (τ) of the large-sized vehicle.

(1) Correction of the amount of emission for soot

The previous section mentions some formulas which is for calculating soot emission from a vehicle in a stable running condition in the tunnel with a height of less than 400 [m] and no longitudinal gradient (traveling speed 40 to 80 [km/hour]). These average value (μ) and standard deviation (σ) of emissions are calculated by the following formula.

$$\mu = \gamma_L \cdot \mu_1 + (1 - \gamma_L) \cdot \mu_2$$

$$\sigma^2 = \gamma_L \cdot \left\{ \sigma_1^2 + (\mu_1 - \mu)^2 \right\} + (1 - \gamma_L) \cdot \left\{ \sigma_2^2 + (\mu_2 - \mu)^2 \right\}$$

μ : Average of the amount of emission for soot [m^2/Km]
σ : Standard deviation of the amount of emission for soot [m^2/Km]
μ_1 : Average of the amount of emission for soot of large vehicles after correction [m^2/Km]
μ_2 : Average of the amount of emission for soot of small vehicles after correction [m^2/Km]
σ_1 : Standard deviation of the amount of emission for soot of large vehicles after correction [m^2/Km]
σ_2 : Standard deviation of the amount of emission for soot of small vehicles after correction [m^2/Km]
γ_L : Large vehicle mixture ratio

The above formula shall use the average value and the standard deviation of the amount of emission for soot shown in the above investigation results. The fine particle rate in soot from each vehicle is also shown in the following table as the result of same investigation.

Table 7-6: Average and Standard deviation of the amount of emission for soot in Japan

Vehicle kind	Amount of emission for soot		Fine particle rate in Soot
	Average	Standard deviation	
Small	0.3 [m^2/km·units]	0.3 [m^2/km·units]	55 [%]
Large	1.5 [m^2/km·units]	0.5 [m^2/km·units]	50 [%]

Source: Road tunnel (Ventilation) technical standard and explanation; Japan Road Association 2010.

The average and the standard deviation after correction of each vehicle are obtained by the following expressions.

$$\mu_1 = \left\{ \mu_L \cdot R_L \cdot K_v \cdot K_L \cdot K_h \right\} + \mu_L \cdot (1 - R_L)$$

$$\sigma_1 = \left\{ \sigma_L \cdot R_L \cdot K_v \cdot K_L \cdot K_h \right\} + \sigma_L \cdot (1 - R_L)$$

$$\mu_2 = \left\{ \mu_S \cdot R_S \cdot K_v \cdot K_L \cdot K_h \right\} + \mu_S \cdot (1 - R_S)$$

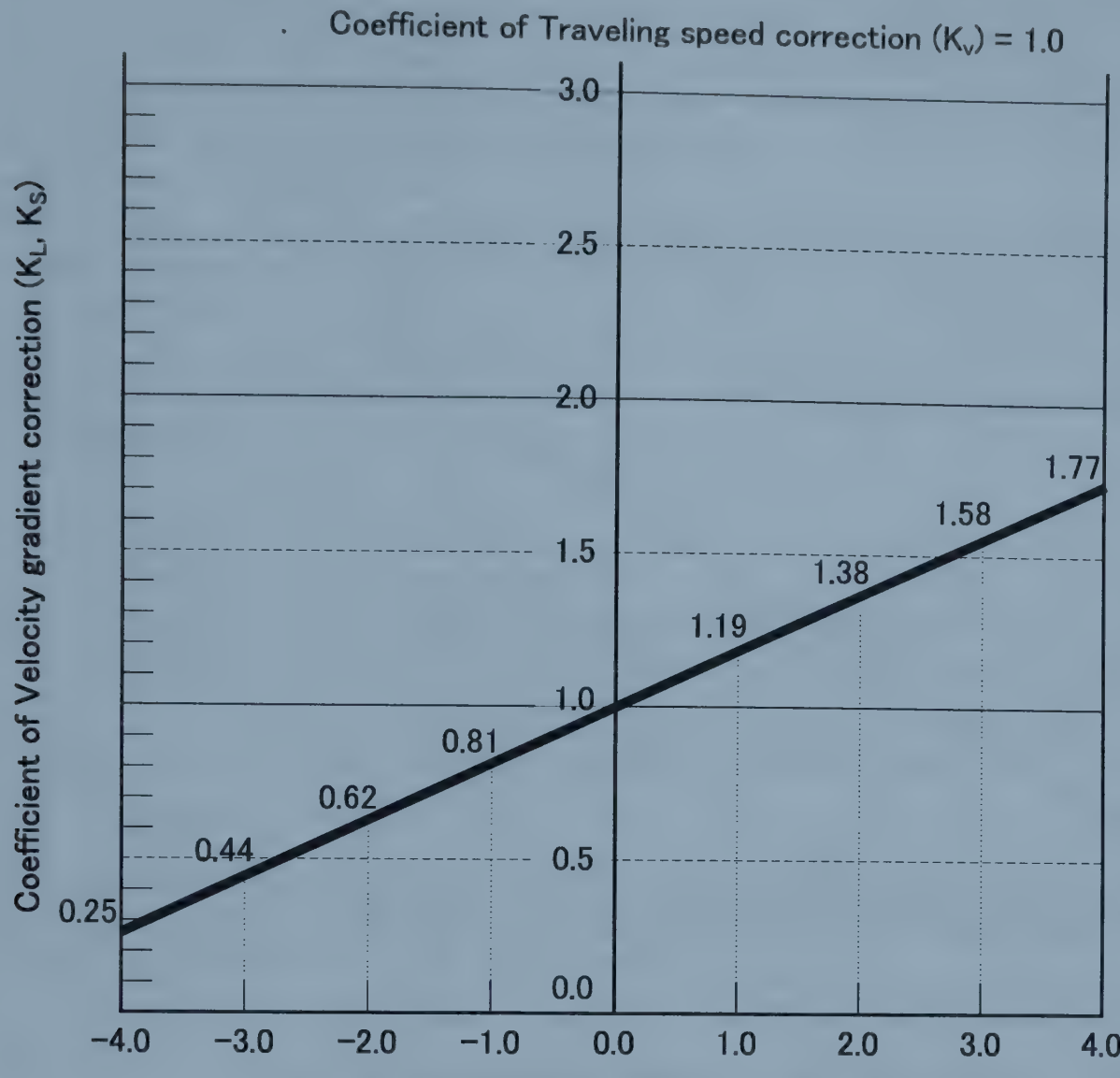
$$\sigma_2 = \left\{ \sigma_S \cdot R_S \cdot K_v \cdot K_L \cdot K_h \right\} + \sigma_S \cdot (1 - R_S)$$

μ_L : Average of the amount of emission for soot of large vehicles [m^2/Km]
μ_S : Average of the amount of emission for soot of small vehicles [m^2/Km]
σ_L : Standard deviation of the amount of emission for soot of large vehicles [m^2/Km]
σ_S : Standard deviation of the amount of emission for soot of small vehicles [m^2/Km]
R_L : Fine particle rate in the soot of large vehicles [%]
R_S : Fine particle rate in the soot of small vehicles [%]
K_v : Coefficient of the traveling speed correction
K_L : Coefficient of the velocity gradient correction of large vehicles
K_S : Coefficient of the velocity gradient correction of small vehicles
K_h : Coefficient of the height correction

(2) Correction coefficient

The soot concentration in automobile exhaust gas has been confirmed to vary depending on the longitudinal gradient of the road and the traveling speed by the experimentally

before. The result of the bench test in 2013 about traveling speed and longitudinal gradient in Japan is shown in the chart below.



Source: Road tunnel (Ventilation) technical standard and explanation; Japan Road Association 2010.

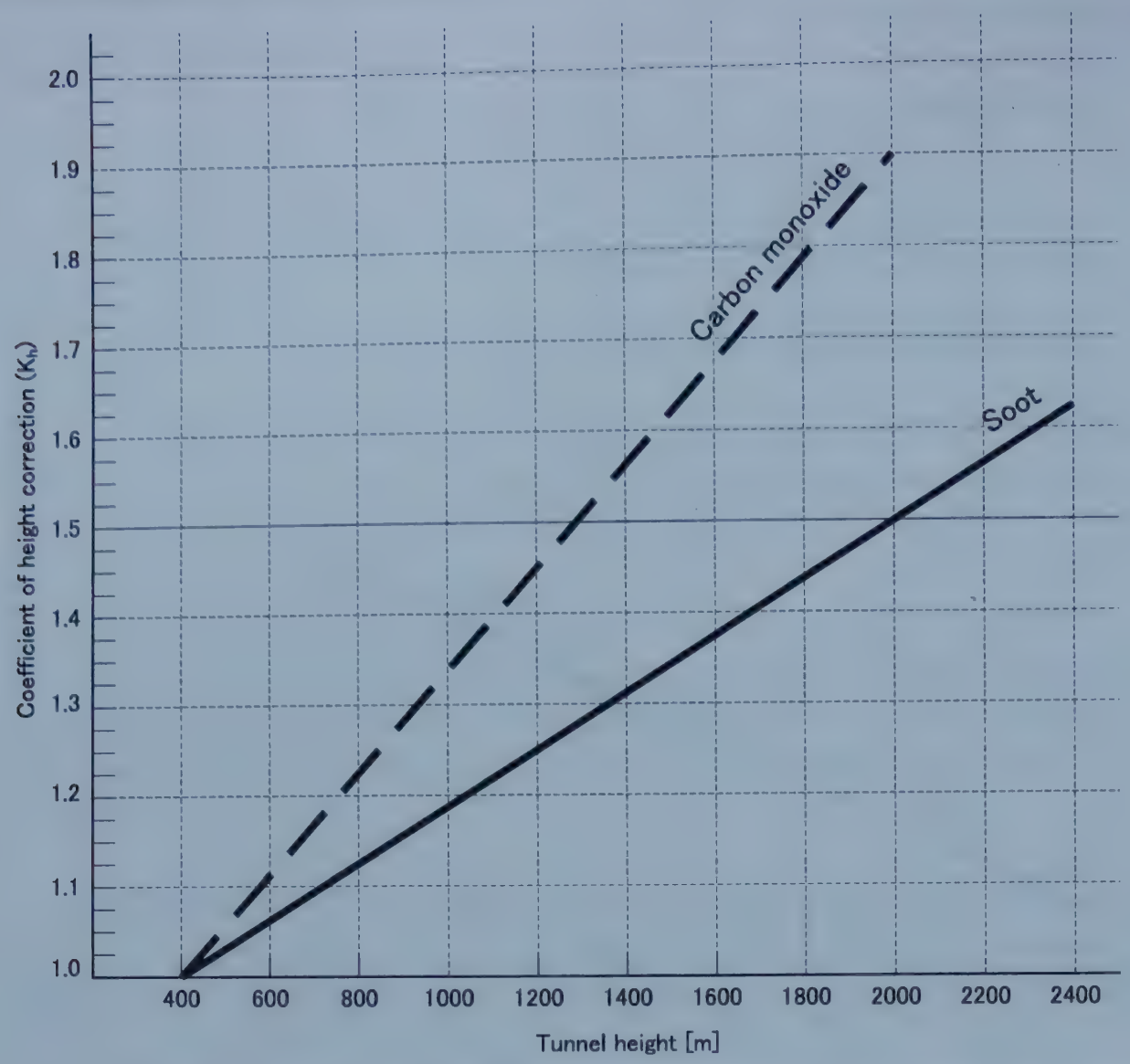
Figure 7-6: Bench test about traveling speed and longitudinal gradient in Japan

The coefficient of traveling speed correction (K_v), the coefficient of velocity gradient correction for large vehicles (K_L), and the coefficient of velocity gradient correction for small vehicles (K_S) are obtained from this chart.

These correction coefficients in 2013 are an experimental result after the harmful substance discharged was stabilized by the exhaust gas regulation in 2005. Thus, the coefficients of velocity gradient correction are the same for large vehicles and small vehicles. And, the coefficient of the traveling speed correction (K_v) was used 1.0 at this time. These correction coefficients had been different from each other before 2013 in Japan.

Incidentally, there is currently no data for correction coefficients with vertical gradients exceeding 4%. Therefore, a special experiment should be required when such longitudinal gradient is used as well.

Emissions of the vehicle are affected by tunnel height also. The results of the actual driving test for the coefficient of height correction (K_h) conducted in Japan are shown below. This correction coefficient (K_h) will be revised again when tunnel ventilation operation in the highland will increase in the future and those ventilation data can accumulate in the future.



Source: Road tunnel (Ventilation) technical standard and explanation; Japan Road Association 2010.

Figure 7-7: Field driving test about height correction in Japan

7.10.3 Ventilation volume for a carbon monoxide

The design value for carbon monoxide (K_{co}) is defined as the average concentration of carbon monoxide because the relationship between exposure time and concentration of carbon monoxide concentration is clear.

$$\underbrace{\frac{\mu}{T_t \cdot Q}}_{\text{Average concentration}} \leqq \underbrace{K_{co} \times 10^6}_{\text{Specified concentration}}$$

μ : Average value of carbon monoxide emissions of vehicle group [m^2 / Km]

By the relational formula relating to the above concentration, the equation of the required ventilation volume (Q) is conducted as follows.

$$Q = \frac{\mu}{T_t \cdot K_{co} \cdot 10^{-6}}$$

(1) Correction of the amount of emission for carbon monoxide

PIARC reports that the correction of carbon monoxide emissions due to longitudinal gradient and traveling speed used in ventilation calculation would be the longitudinal

gradient ranges from + 6% to -6% and the traveling speed correction coefficient ranges from 0.7 to 1.3.
On the other hand, the Japanese investigation, could not find a clear relationship on the correction of carbon monoxide emissions by longitudinal gradient and traveling speed. Therefore, the correction of carbon monoxide emission by longitudinal gradient and traveling speed for calculating ventilation volume is not carried out in Japan. The correction will be applied to the calculation of the ventilation volume when this relationship becomes clear.
The carbon monoxide emission should be required to consider the effect of tunnel height as same as the case of the soot emission. The results of the actual driving test for the coefficient of height correction (K_h) conducted in Japan are shown in the above charts. This correction coefficient (K_h) will be revised again when tunnel ventilation operation in the highland will increase in the future and those ventilation data can be accumulated in the future.

$$\mu_0 = K_h \cdot \mu$$

K_h : Coefficient of Height correction
u_0 : Average of the amount of emission for carbon monoxide after correction [m^3/Km]
u : Average of the amount of emission for carbon monoxide [m^3/km]

7.10.4 The calculation for the required ventilation volume

(1) General

The required ventilation volume should be calculated using the following equation.
q in this equation is defined as unit ventilation and is the amount of ventilation per 1 [units/hour] per 1000 [m] of tunnel length.

$$Q_{rq} = q \cdot N \cdot L_r \cdot 10^{-3}$$

(2) The unit amount of emission for soot (q)

The unit amount of emission for soot (q) is the amount of emission for soot when 1 [km] of traveling distance in 1 [units/hour] of traffic volume with respect to the above calculation formula for the amount of emission for soot.

$$q = \left\{ \frac{3\sigma + \sqrt{9\sigma^2 + 8\mu \cdot K_{VI} \cdot A_r \cdot 1000}}{60 \cdot \sqrt{8 \cdot A_r \cdot 1000} \cdot K_{VI}} \right\}^2$$

(3) The unit amount of emission for carbon monoxide (q)

The unit amount of emission for carbon monoxide is the amount of emission for carbon monoxide when 1 [km] of traveling distance in 1 [units/hour] of traffic volume with respect to the above calculation formula for the amount of emission for carbon monoxide.

$$q = \frac{\mu}{T_t \cdot K_{co} \cdot 10^{-6}}$$

7.10.5 Concentration distribution

The concentration (K) in the tunnel is expressed as the ratio of the amount of harmful substance (μ) discharged in the tunnel and the amount of fresh air (Q_0).

$$K = \frac{\mu'}{Q_o}$$

Generally, the concentration distribution is determined as a concentration ratio where the amount of emission for harmful substances from the vehicle is kept constant, only the amount of emission due to the gradient is corrected, and the design concentration is set to 1.0.

$$\frac{K}{K_o} = \frac{Q_{rq}}{Q}$$

Q_{rq} : Required ventilation volume [m^3]
Q : Fresh air volume [m^3]
K_0 : Design concentration [m^3]

In the case of carbon monoxide, the concentration ratio to concentration can be easily converted. However, in the case of soot, the distribution is different from the distribution of the transmittance because the concentration distribution is created for the optical density. The concentration distribution varies depending on how fresh air is supplied (ventilation system).

7.11 Required ventilation capability

The required ventilation capacity must consider the following points first whether it is ensured by natural ventilation. Because there is a cost impact for the mechanical ventilation and the natural ventilation limits for eliminating the harmful substance from the tunnel.

7.11.1 Limits of Natural ventilation

Harmful substances are eliminated from the tunnel by the wind (natural ventilation) passing through inside the tunnel. This eliminating phenomenon can be explained by Bernoulli's theorem. According to Bernoulli's theorem, the dynamic pressure and the static pressure distributes in the tunnel by the wind (ventilation) passing through inside the tunnel.
This dynamic pressure can be ignored in the calculation because it is very little. The reason is that the fluid density of the air is extremely small, and the wind speed used in the ventilation calculation is less than 1/10 of the speed of sound, and the compressibility of air can be neglected.
Thus, the calculation of ventilation wind pressure is able to basically be performed using Bernoulli's theorem and the law of momentum. And, these pressures are the resistance against the ventilation of the tunnel, which is the sum of the pressure losses.
The necessity of a mechanical ventilation system can be judged by whether this ventilation resistance satisfies the ventilation volume required to properly eliminate harmful substances from the tunnel.
Moreover, this natural ventilation is composed of natural wind (U_n) caused by pressure differential, temperature difference, etc., and roadway wind (U_r) caused by piston action of the vehicle traffic. And, Natural ventilation capability is ventilation power by natural wind (U_n), and traffic ventilation capacity is ventilation power by roadway wind (U_r). Thus, the required ventilation capability is carried out by the calculation of the natural ventilation capability, the traffic ventilation capability, and the mechanical ventilation capability.

7.11.2 Natural wind and Natural ventilation capability

(1) Natural wind (U_n)

A natural wind (U_n) is generated by the pressure difference in which the following phenomena are interrelated and operate. Also, these phenomena are not factors that can be separated independently.

- The meteorological pressure differential between tunnel portals
- Pressure difference based on temperature distribution inside and outside the tunnel
- The damming pressure difference occurs when the natural wind moves the inside from the outside of the tunnel.

(2) Natural ventilation capability ($\Delta P_{M\tau}$)

Natural ventilation capability ($\Delta P_{M\tau}$) is ventilation power by natural wind (U_n).

$$\Delta P_r = (1+\zeta_e+\lambda \cdot \frac{L}{D_r}) \cdot \frac{\rho}{2} \cdot U_r^2$$

$\Delta P_{M\tau}$: Natural ventilation capability [P_a]
ζ_e : The tunnel inlet loss factor
λ : Tunnel wall friction loss coefficient
L : Tunnel length [m]
D : Tunnel representative diameter [m]
U_n : Natural wind speed [m/s]
ρ : Air density [kg/m^3]

In fact, the natural ventilation capability ($\Delta P_{M\tau}$) in the tunnel does not always exist, and even if it exists, the direction of the natural wind blowing changes according to the time, temperature, and atmospheric pressure, and it always fluctuates. The value of actual natural ventilation capability ($\Delta P_{M\tau}$) must be measured after tunnel penetration.
In the case of tunnels in general topographical conditions and there are no data such as survey results, natural wind (U_n) is defined 2.5 [m/s] as a reference.

7.11.3 Roadway wind and Traffic ventilation capability

(1) Roadway wind (U_r)

A roadway wind (U_r) is generated in the tunnel when the piston action by the vehicle traffic occurs in the tunnel.

(2) Traffic ventilation capability (ΔP_T)

Traffic ventilation capability (ΔP_T) is often relatively large compared to natural ventilation.
In a one-way tunnel, even with a considerably long tunnel, when the flow of the vehicle is smooth, there is a case that sufficient ventilation can be performed only by the traffic ventilation capability. However, in a two-way tunnel, the traffic ventilation capability would cancel each other out, acting in reverse as a resistance, so even with relatively short tunnels, mechanical ventilation may be required.
Generally, the traffic ventilation capability due to the piston action of the vehicle traveling in the tunnel can be obtained in the steady-state by the following equation.

$$\Delta P_t = \frac{A_m}{A_r} \cdot \frac{\rho}{2} \cdot n_+ \cdot (V_{t+} - U_r)^2 - \frac{A_m}{A_r} \cdot \frac{\rho}{2} \cdot n_- \cdot (V_{t-} + U_r)^2$$

The value for n_+, n_-, V_{t+}, and V_{t-} are based on the direction of the wind speed (U_r) in the tunnel.

ΔP_τ : Traffic ventilation capability [P_a]
U_r : Roadway wind speed [m/s]
n_+ : Number of the vehicle with the same direction to direction of roadway wind [units]
n_- : Number of the vehicle with opposite direction to direction of roadway wind [units]
A_m : Vehicle equivalent resistance area [m^2]
A_r : Roadway cross-section area [m^2]
V_{r+} : Velocity of the vehicle with the same direction to direction of roadway wind [m/s]
V_{r-} : Velocity of the vehicle with opposite direction to the direction of roadway wind [m/s]

(3) Traffic ventilation volume (Q_r)

The traffic ventilation volume (Q_r) of the roadway wind speed (U_r) generated in the tunnel is the product of the roadway cross-section area (A_r).

$$Q_r = U_r \cdot A_r$$

Q_r : Traffic ventilation volume [m^3]
U_r : Roadway wind speed [m/s]
A_r : Roadway cross section area [m^2]

(4) Numbers of the vehicle in the tunnel (n)

The numbers of vehicles (n) traveling in the tunnel are the number of vehicles when the vehicles are traveling in the tunnel by the design traffic volume and in case that there are no more vehicles entering newly. The number of the vehicle (n) is calculated by the following formula by design traffic volume (N), tunnel length (L), the vehicle traveling speed (V_t).

(One-way tunnel)

$$n = \frac{N \cdot L}{1000\ V_t}$$

(Two-way tunnel)

$$n = \frac{N \cdot L}{1000\ V_t} \cdot \frac{1}{2}$$

n : Number of the vehicle in the tunnel [units]
L : Tunnel length [km]
N : Hourly traffic volume [units/hour]
V_t : Velocity of vehicle [km/hour]

7.11.4 Vehicle equivalent resistance area

Vehicle equivalent resistance area (A_m) is defined by the product of the front projected area (A_c) of the vehicle and the resistance coefficient of the vehicle (ζ_c). These vehicles in this tunnel would consist of large vehicles and small vehicles. The front projected

area of these kinds of the vehicle is different because the front projected area is depending upon the shape of the vehicle. Vehicle equivalent resistance area (A_m) is calculated as the average value using the equivalent resistance area averaged in each of the large and small vehicles and the large vehicle mixture ratio (γ_L).

$$A_m = (1-\gamma_L) \cdot (A_c \cdot \xi_c)_s + \gamma_L \cdot (A_c \cdot \xi_c)_L$$

γ_L : Large vehicle mixture ratio
A_c : The front projected area
ξ_c : The resistance coefficient
$(A_c \cdot \xi_c)_s$: Equivalent resistance area of a small car
$(A_c \cdot \xi_c)_L$: Equivalent resistance area of large vehicle

The front projected area (A_c) of the vehicle is organized for each type of vehicle, as shown in the table below.

Table 7-7: The front projected area

Large size vehicle	Small size vehicle
6.88 [m^2]	2.45 [m^2]

Source: Road tunnel (Ventilation) technical standard and explanation; Japan Road Association 2010.

The resistance coefficient (ξ_c) of the vehicle differs depending on the shape of the vehicle and the front projected area (A_c). Therefore, the resistance coefficient (ξ_c) is determined by survey and experiment, and the resistance coefficient (ξ_c) of the vehicle is as follows, up to 25% occupancy of a large car and up to 15% occupancy of a small car.

$$\xi_{CL} = 0.0309x + 0.724$$
$$\xi_{CS} = 0.0140x + 0.308$$

ξ_{CL} : Car resistance coefficient of a large car
ξ_{CL} : Car resistance coefficient of a small car
x : Occupancy rate [%]

As a result of the above, the following formula is obtained by these above formulas and the values of Table 7-4.

$$A_m = 0.78 + \frac{9.1}{A_r} + (4.21 + \frac{137}{A_r}) \cdot \gamma_L$$

7.11.5 Traffic ventilation diagram

The traffic ventilation diagram is used to explain the pressure rise inside the tunnel caused by the roadway wind (U_r) and the natural wind (U_n). The pressure rise (ΔP_r) in the tunnel is the sum of the traffic ventilation capability (ΔP_t) and the natural ventilation capability (ΔP_{MT}).

$$\Delta P_r = \pm \Delta P_t \pm \Delta P_{MT}$$

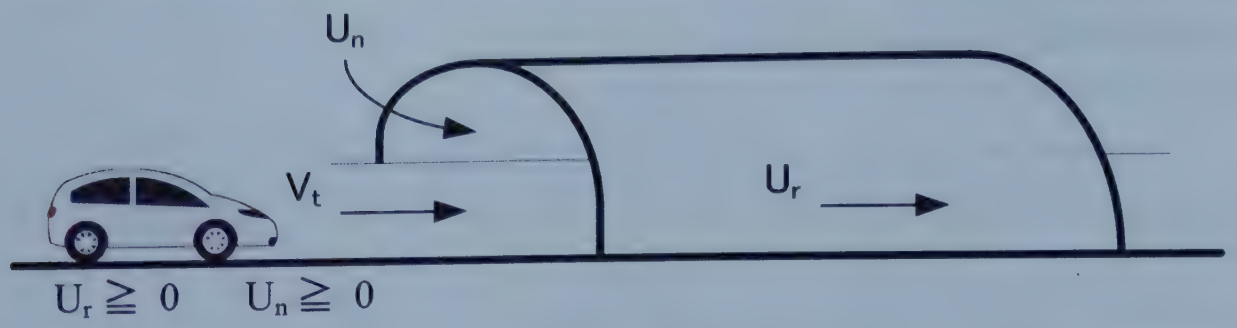
In addition, since this pressure rise (ΔP_r) inside the tunnel shall be the ventilation resistance in the roadway wind (U_r), it is also shown by the following formula.

$$\Delta P_r = (1 + \xi_e + \lambda \cdot \frac{L}{D_r}) \cdot \frac{\rho}{2} \cdot U_r^2$$

7.11.6 Traffic ventilation diagram for one way traffic

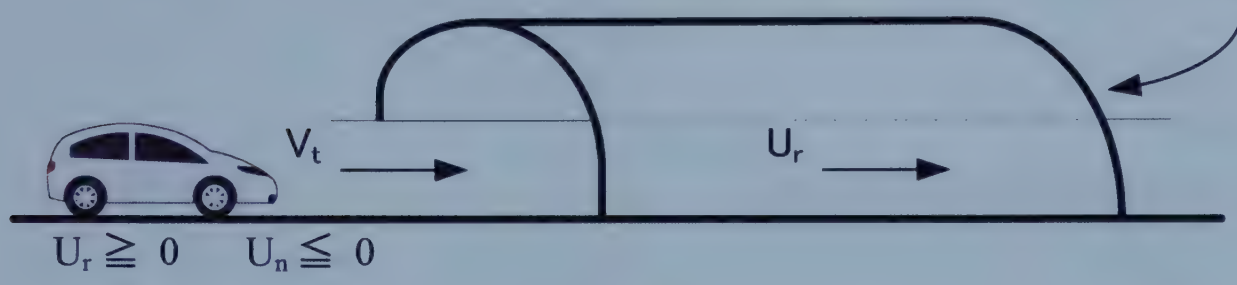
The above relational formula differs depending on the direction of the wind inside the tunnel, the direction of the roadway wind (U_r), and the direction of the natural wind (U_n).

(1) In case of that, the direction of wind inside the tunnel is the same direction of the roadway wind (U_r) and the natural wind (U_n).



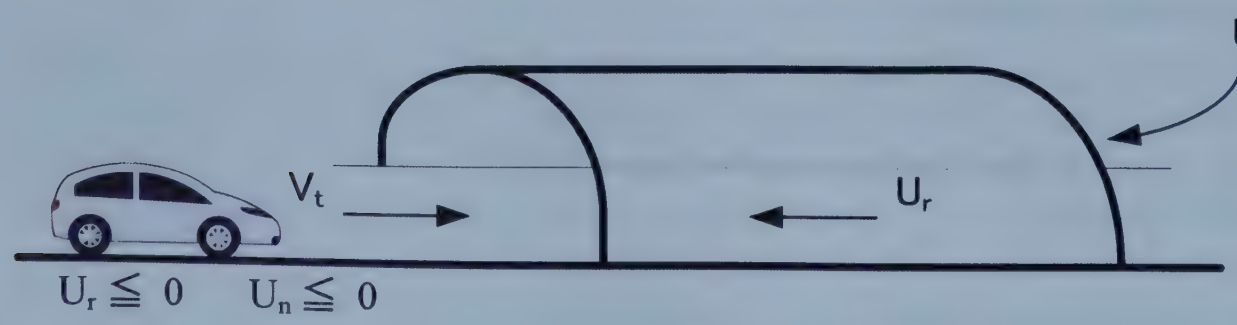
$$\underbrace{(1+\zeta_e+\lambda\cdot\frac{L}{D})_r\cdot\frac{\rho}{2}\cdot U_r^2}_{\text{Positive wind}}=\underbrace{\frac{A_m}{A_r}\cdot\frac{\rho}{2}\cdot n\cdot(V_t--U_r)^2}_{\text{Pressurization}}+\underbrace{(1+\zeta_e+\lambda\cdot\frac{L}{D})_r\cdot\frac{\rho}{2}\cdot U_n^2}_{\text{Forward pressure}}$$

(2) In case that the direction of wind inside the tunnel is the same direction of the roadway wind (U_r), and is the opposite direction of the natural wind (U_n).



$$\underbrace{(1+\zeta_e+\lambda\cdot\frac{L}{D})_r\cdot\frac{\rho}{2}\cdot U_r^2}_{\text{Positive wind}}=\underbrace{\frac{A_m}{A_r}\cdot\frac{\rho}{2}\cdot n\cdot(V_t--U_r)^2}_{\text{Pressurization}}+\underbrace{(1+\zeta_e+\lambda\cdot\frac{L}{D})_r\cdot\frac{\rho}{2}\cdot U_n^2}_{\text{Reverse pressure}}$$

(3) In case that the direction of wind inside the tunnel is the same direction of the natural wind (U_n), and is the opposite direction of the roadway wind (U_r).



$$\underbrace{(1+\zeta_e+\lambda\cdot\frac{L}{D})_r\cdot\frac{\rho}{2}\cdot U_r^2}_{\text{Negative wind}}=\underbrace{-\frac{A_m}{A_r}\cdot\frac{\rho}{2}\cdot n\cdot(V_t+U_r)^2}_{\text{Resistance}}\underbrace{--(1+\zeta_e+\lambda\cdot\frac{L}{D})_r\cdot\frac{\rho}{2}\cdot U_n^2}_{\text{Reverse pressure}}$$

Source: Road tunnel (Ventilation) technical standard and explanation; Japan Road Association 2010.

Figure 7-8: Traffic ventilation diagram for one way traffic

(4) Determination of ventilation method

The traffic ventilation volume (Q_r) should be calculated by the roadway wind (U_r) obtained in the above formulas (1) to (3) and the roadway cross-section area (A_r).
By based on the comparison of the calculated traffic ventilation volume (Q_r) and the calculated required ventilation volume (Q_{rq}), it can be judged whether mechanical ventilation is required.

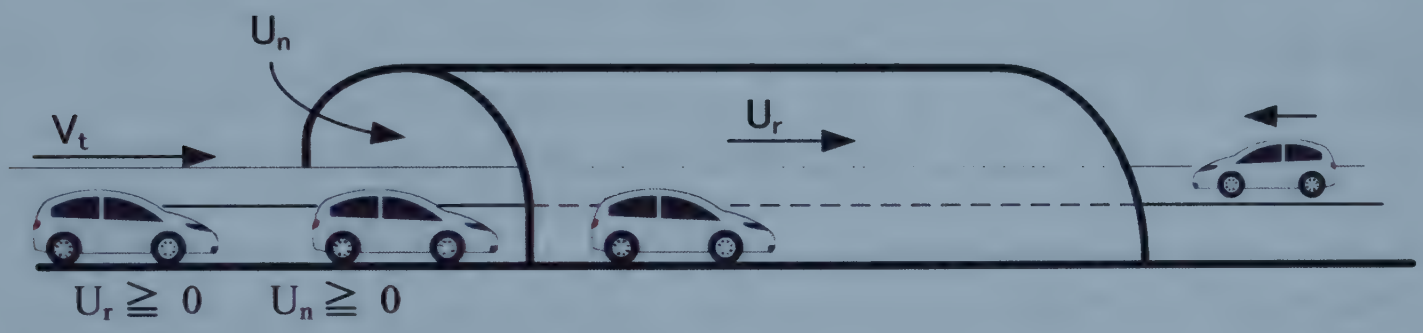
$Q_{rq} \leqq Q_r$: Mechanical ventilation must be required

$Q_{rq} > Q_r$: Mechanical ventilation won't be required

7.11.7 Traffic ventilation diagram for two way traffic

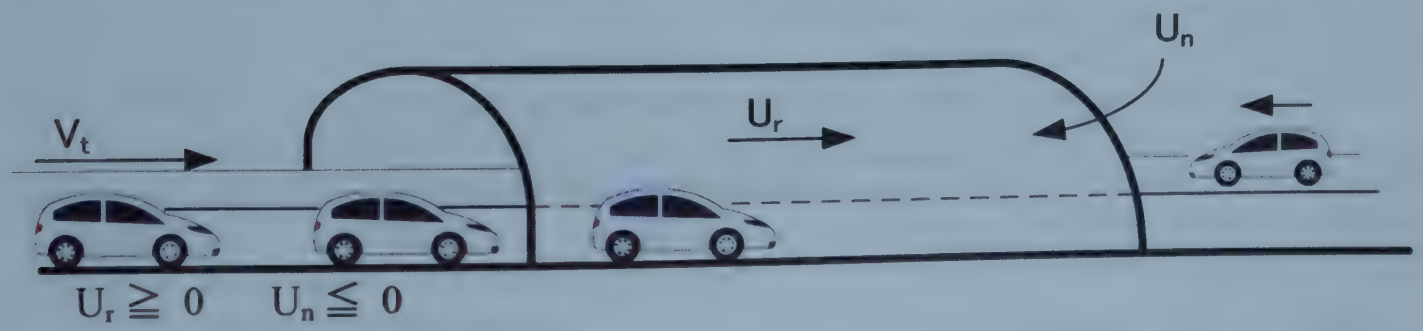
There would be a traffic ventilation wind (U_r) even if in case that there is a natural wind (U_n) and a large difference in traffic volume between the upward direction and the downward direction in two-way traffic.
Traffic in a direction with many numbers of traffic volume is defined as n_+ and the traffic ventilation wind (U_r) in that direction is positive (forward wind), the following calculation formulas can be derived by applying the similar calculation formula of one-way traffic.

(1) In case of that, the direction of wind inside the tunnel is the same direction of the roadway wind (U_r) and the natural wind (U_n). (The direction with many traffic volumes should be defined as a positive direction.)



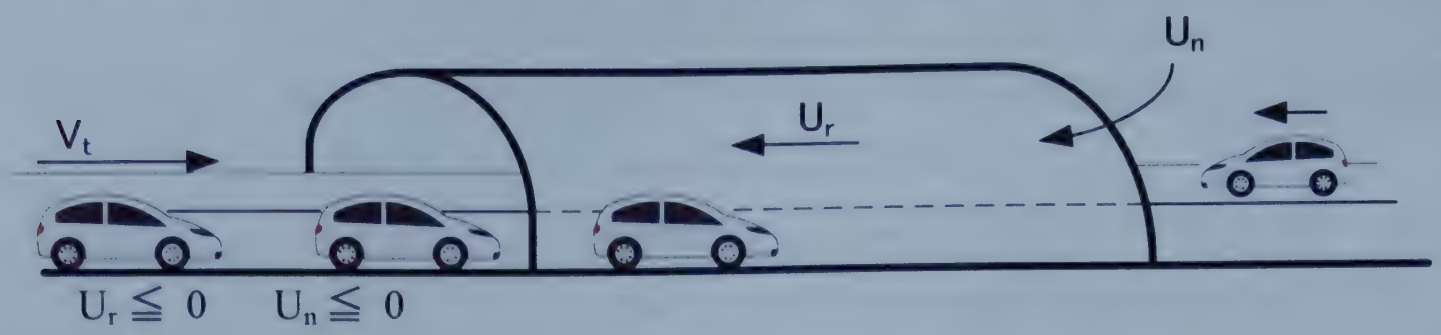
$$\underbrace{(1+\zeta_e+\lambda\cdot\frac{L}{D})_r\cdot\frac{\rho}{2}\cdot U_r^2}_{\text{Positive wind}}=\frac{A_m}{A_r}\cdot\frac{\rho}{2}\cdot\{\underbrace{n_+\cdot(V_t-U_r)^2}_{\text{Pressurization}}-\underbrace{n_-\cdot(V_t+U_r)^2}_{\text{Resistance}}\}+\underbrace{(1+\zeta_e+\lambda\cdot\frac{L}{D})_r\cdot\frac{\rho}{2}\cdot U_n^2}_{\text{Forward pressure}}$$

(2) In case that the direction of wind inside the tunnel is the same direction of the roadway wind (U_r), and is the opposite direction of the natural wind (U_n). (The direction with many traffic volumes should be defined as a positive direction.)



$$\underbrace{(1+\zeta_e+\lambda\cdot\frac{L}{D})_r\cdot\frac{\rho}{2}\cdot U_r^2}_{\text{Positive wind}}=\frac{A_m}{A_r}\cdot\frac{\rho}{2}\cdot\{\underbrace{n_+\cdot(V_t-U_r)^2}_{\text{Pressurization}}-\underbrace{n_-\cdot(V_t+U_r)^2}_{\text{Resistance}}\}-\underbrace{(1+\zeta_e+\lambda\cdot\frac{L}{D})_r\cdot\frac{\rho}{2}\cdot U_n^2}_{\text{Reverse pressure}}$$

(3) In case that the direction of wind inside the tunnel is the same direction of the natural wind (U_n), and is the opposite direction of the roadway wind (U_r). (The direction with many traffic volumes should be defined as a positive direction.)



$$\underbrace{(1+\zeta_e+\lambda\cdot\frac{L}{D})_r\cdot\frac{\rho}{2}\cdot U_r^2}_{\text{Negative wind}} = -\frac{A_m}{A_r}\cdot\frac{\rho}{2}\cdot\underbrace{\{n_+\cdot(V_t+U_r)^2}_{\text{Resistance}} \underbrace{-n_-\cdot(V_t-U_r)^2\}}_{\text{Pressurization}} + \underbrace{(1+\zeta_e+\lambda\cdot\frac{L}{D})_r\cdot\frac{\rho}{2}\cdot U_n^2}_{\text{Reverse pressure}}$$

Source: Road tunnel (Ventilation) technical standard and explanation; Japan Road Association 2010.

Figure 7-9: Traffic ventilation diagram for two way traffic

(4) Traffic ventilation capability (ΔP_T)

In case that the upward and downward traffic volume ratios are 50:50 respectively, the relational formula is as follows. The value for n_+, n_-, V_{t+}, and V_{t-} are based on the direction of roadway wind (U_r) in the tunnel as same as the previous formula.

$$\Delta P_t = \frac{A_m}{A_r}\cdot\frac{\rho}{2}\cdot\{n_+\cdot(V_t-U_r)^2 - n_-\cdot(V_t+U_r)^2\}$$

$$= \frac{A_m}{A_r}\cdot\frac{\rho}{2}\cdot\frac{n_+\cdot L}{1000\,V_{t+}}\cdot(V_{t+}-U_r)^2 - \frac{A_m}{A_r}\cdot\frac{\rho}{2}\cdot\frac{n_-\cdot L}{1000\,V_{t-}}\cdot(V_{t-}+U_r)^2$$

ΔP_T : Traffic ventilation capability [P_a]
U_r : Roadway wind speed [m/s]
n_+ : Number of the vehicle with the same direction to direction of roadway wind [units]
n_- : Number of the vehicle with opposite direction to direction of roadway wind [units]
N_+ : Hourly traffic volume with the same direction to direction of roadway wind [units /hour]
N_- : Hourly traffic volume with opposite direction to the direction of roadway wind [units/hour]
L : Tunnel length [km]
A_m : Vehicle equivalent resistance area [m^2]
A_r : Roadway cross-section area [m^2]
V_{r+} : Velocity of the vehicle with the same direction to direction of roadway wind [m/s]
V_{r-} : Velocity of the vehicle with opposite direction to the direction of roadway wind [m/s]

7.12 Measuring equipment

The measuring equipment of the ventilation facility should be considered simultaneously with the design of tunnel length, structure, ventilation system, etc. The measuring equipment should be installed while considering the requirement even if dedicated

equipment for smoke evacuation is installed and the equipment for countermeasures against traffic jams is installed. The typical measuring equipment used in a ventilation facility is shown below.

- Visibility meter (VI meter) : Soot and smog transmittance measuring device
- Carbon monoxide analyzer (CO meter) : Carbon monoxide detecting device
- Air flow Velocity and direction meter (AV meter) : Wind vane and anemometer device
- Traffic counter (TC meter) : Traffic measurement device
- Atmospheric pressure gage (AP meter) : Apparatus for measuring natural wind speed in the tunnel

7.12.1 Installation

The measuring equipment shall determine the location of installation considering the effective and efficient operation of ventilation. Moreover, measuring equipment shall be installed in consideration of ventilation system, transportation system, ventilation equipment scale, ventilation control, etc.
The basic installation conditions of the measuring equipment are shown below.

a) Visibility Meter and Carbon monoxide analysis should be installed in places where the maximum contamination concentration can be detected under normal conditions.
b) Visibility meter and Carbon monoxide analysis should also be installed in places where the maximum contamination concentration can be detected in the two-way traffic tunnels when the wind direction inside the tunnel is reversed.
c) Air flow velocity and direction meter should be installed at least one for one ventilation section separated by shaft and well-head of the tunnel.
d) Atmospheric pressure gage should be installed when the natural wind is large and the economy of ventilation operation can be expected with the atmospheric pressure (natural ventilation) controlled.

7.12.2 Layout

The measuring equipment should be installed in the place where it can ascertain the behavior of air inside the tunnel as precisely as possible. The layout of the measuring equipment also shall consider the influence of the wind from the ventilator, the influence of the outside air, the lighting from the outside, the influence of the curving road, etc.

7.12.3 Installation for Visibility meter

Installation for the Visibility meter should be in compliance with the following in principle.

(1) Layout

(a) On one side of the side wall on the driving, lane side should be installed.
(b) The installation distance should ensure the distance at least 7 times the tunnel typical diameter from the tunnel mouth, the electric dust collecting chamber, the pit of the tunnel, the ventilation shaft, and the Jet-fan due to avoid the influence of tunnel inside turbulence.
(c) The receiver unit should be installed where it is not be affected by natural lighting from the tunnel mouth.

(d) The irradiation unit should be installed in a place where it ensures a sufficient separation distance from the receiver unit because the wall reflection lighting from the irradiated unit protects to enter into the receiver unit.
(e) The installation distance between the irradiation unit and the receiver unit should be ensured 100 [m] in principle.

(2) Height

The height of installation should be ensured 4.7 to 5.0 [m] from the road surface, or 2.2 to 2.5 [m] from the tunnel monitor passage or the tunnel inspection gallery.

7.12.4 Installation for Carbon monoxide analyzer

Installation for the Carbon monoxide analyzer should be in compliance with the following in principle.

(1) Layout

(a) Install on one side of the side wall on the driving lane side.
(b) The installation distance should ensure the distance at least 7 times the tunnel typical diameter from the tunnel mouth, the electric dust collecting chamber, the pit of the tunnel, the ventilation shaft, and the Jet-fan due to avoid the influence of the tunnel inside turbulence.
(c) The installation of the irradiation unit and the receiver unit should be near the middle of the tunnel.
(d) The installation should be in the tunnel monitor passage side or in the place where lots of other equipment are installed for facilitating maintenance.

(2) Height

The installation should be installed inside the box-out on the tunnel monitor passage or the tunnel inspection gallery.

7.12.5 Installation for Air flow velocity and direction meter

Installation for Air flow velocity and direction meter should be in compliance with the following in principle.

(1) Layout

(a) Install on one side of the side wall on the driving lane side.
(b) The installation of the following places must be restricted except in unavoidable circumstances.

- The place where the distance is less than 7 times the tunnel typical diameter from the tunnel mouth, the electric dust collecting chamber, the pit of the tunnel, the ventilation shaft, and the Jet-fan.
- Emergency parking zone
- The place where the distance is less than 150 [m] of leeward from Jet-fan and/or Air flow nozzle.
- In the vicinity of a structure (For example, the road sign board) that becomes an obstacle against the flow of ventilation wind.

(2) Height

The height of installation should be ensured 4.7 to 5.0 [m] from the road surface, or 2.2 to 2.3 [m] from the tunnel monitor passage or from the tunnel inspection gallery to the bottom part of the detector unit.

7.13 Operation and control

Operation and control for ventilation facility can carry out the operation of the ventilator effectively and economically for soot concentration and carbon monoxide concentration inside the tunnel. And, Ventilation control can carry out the operation of the ventilator with minimal power consumption to ensure target values for the visual environment and air pollution in the tunnel. Moreover, the operation corresponding to the occurrence event can be switched when a fire occurs in the tunnel or there is an abnormality such as a traffic jam.

7.13.1 System

The system of operation and control for ventilation facility must select appropriate control methods singly or in combination according to the tunnel ventilation method, equipment size and etc. And, Ventilation control mainly corresponds to normal traffic however it is also required to cope up with the occurrence of congestion, accident, and fire. Moreover, the Ventilation control system is based on Automatic control in principle and enables manual control as well. The Ventilation control method is shown below.

- Feedback control
- Feedforward control
- High-grade control
- Program control

7.13.2 Normal operation

(1) Isolated control

Isolated control is a control to operate and stop ventilation units one by one for inspection and trial operation. An interlock system between the ventilation unit and the accessory unit should be required in this control.

(2) Automatic control

The automatic control system as shown below must be carried out the operation singly or in combination according to the tunnel ventilation method and equipment size etc.

(a) Feedback control
In order to ensure the reference concentration of contamination of the carbon monoxide and transmittance of light in the tunnel, the ventilation facility is controlled the ventilation volume so that the difference between the measured value and the reference target value is minimized.

(b) Feedforward control
The flow of roadway wind is predicted and estimate the amount of exhaust gas generated based on that prediction. The ventilation volume according to that exhaust gas amount is calculated, and a correction based on the existing contamination state is further performed to determine the ventilation volume.

(c) High-grade control
By using process data (For example, the concentration of contamination inside the tunnel, the traffic volume) as input values, the judgment of ventilation operation and

prediction of contamination are controlled by using AI and fuzzy logic. This ventilation control should be applied in which multiple ventilators (Jet-fan, the sending/discharging aircraft, and the dust collector) are installed and multiple ventilation sections are provided.

(d) Program control
In the case where a clear concentration of contamination inside the tunnel and traffic volume pattern is obtained, the operation of ventilation for each time period is controlled in advance with a specified amount of ventilation as a program. This control method should be usually constructed in combination with measurement control (FB control, FF control, High-grade control).

(3) Manual control

Manual control is used for backup of Automatic control system and breakdown of ventilation equipment etc. And, Manual control is controlled the ventilation volume by manually driving ventilation equipment.

7.13.3 Emergency operation

(1) Manual control for fire

The Operation method corresponding to several assumed fires should be set before, the operated by choosing one suitable operation method from these settings when a fire occurs.

(2) Automatic control for fire

The automatic control for fire automatically switches to the fire operation by the signal from the fire alarm. This control has the program control and the velocity control for making slow flow.

(3) Program control

In case of fire in the tunnel, the ventilator has to be stopped when continuing the ventilation operation can be harmful to people's evacuation or standby inside. However, the emergency ventilation control shall be carried out when the operating ventilation facility in a certain pattern can be effective for people evacuation and standby inside.

(4) Velocity control for making slow flow

Simultaneously with the detection of the fire, the ventilation operation is controlled based on the traffic condition and the measurement result of the wind speed so that the wind speed in the roadway of the fire section is lowered.
By this control to be measured the wind speeds in the roadway and operated the ventilator to cancel the traffic ventilation capability and the natural ventilation capability, and sustains the wind speed in the roadway section of the fire area at low wind speed. Also, two or more AV meters shall be installed in one ventilation section.

7.14 Design

7.14.1 General

In the design of the ventilation facility, the following items should be examined at each stage of stages the basic planning, the determining for tunnel cross-section and duct cross-section, and the design of ventilation facility specifications. Each content and

design accuracies in each below item would be different depending on the contents required at each above stage.

- Calculation of Required ventilation volume
- Calculation of Required ventilation capability
- Design of mechanical ventilation
- Design of ventilation system, ventilation duct
- Design of control unit for the ventilator, design of ventilator
- Operation of the ventilator, other

The design for the ventilation duct and ventilation facilities can be carried out after the tunnel specifications, the required ventilation volume, and the ventilation method are determined. The dynamic pressure and static pressure necessary for the ventilation facility, specifications, and quantity can be designed by tunnel parameters and ventilation specifications after the ventilation method are decided.
At the final stage of the design, calculate the accuracy of the dynamic pressure and the static pressure by clarifying the structural size and shape of the ventilation facility. In addition, the operating characteristics, taking into consideration the finalization of ventilation specifications, prediction of various conditions (wind speed, static pressure distribution) during ventilation operation, and load fluctuation are examined.

7.14.2 The linear ventilation system in a mountain road tunnel

The traffic volume is small compared to urban road tunnels so that the linear ventilation system is the most adoptable ventilation system in a mountain road tunnel. Moreover, this natural ventilation capacity is high as well.
In the linear ventilation system design, it should be designed for adding these values which the traffic ventilation capability (ΔP_τ) becomes a resistance or a propulsive force depending on the traffic system.

(1) Traffic ventilation capability (ΔP_T)

Traffic ventilation capability (ΔP_τ) is shown formula as follows.

$$\Delta P_t = \frac{A_m}{A_r} \cdot \frac{\rho}{2} \cdot \{n_+ \cdot (V_t - U_r)^2 - n_- \cdot (V_t + U_r)^2\}$$

(2) Ventilation resistance (ΔP_r)

Ventilation resistance (ΔP_r) is shown formula as follows.

$$\Delta P_r = (1+\zeta_e+\lambda \cdot \frac{L}{D_r}) \cdot \frac{\rho}{2} \cdot U_r^2$$

(3) Natural ventilation capability (ΔP_{MT})

In ventilation design, the pressure (natural ventilation capability) inside the tunnel caused by natural wind, temperature, pressure differential, etc. is used as resistance. This natural ventilation capability is expressed by the following equation using the wind speed of natural wind inside the tunnel.

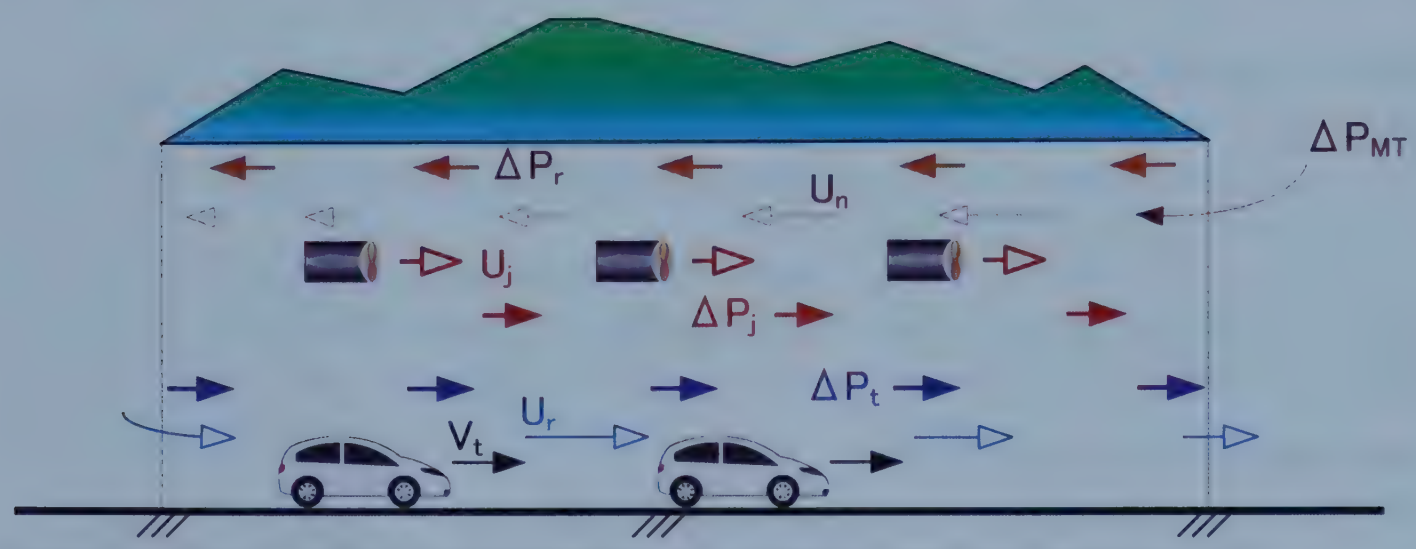
$$\Delta P_r = (1+\zeta_e+\lambda \cdot \frac{L}{D_r}) \cdot \frac{\rho}{2} \cdot U_r^2$$

(4) Mechanical ventilation capability (pressure rise inside the tunnel)

The mechanical ventilation capability in the linear ventilation system is implemented by either method, including jet fans, air supply nozzles, ceiling boards, etc.
Generally, the method of air supply nozzle is concerned with a part of the tunnel body design and would be long construction period, high cost, difficult the post-construction as well. However, the largest ventilation capacity can be secured among their methods.
On the other hand, the method of Jet-fan would be inferior in ventilation capability to other methods. However, it has advantages of the construction period, cost, and post-construction.
The ventilation volume calculation method for them is explained in the next section.

7.14.3 Jet-fan type of mechanical ventilation

The Jet-fan feature is to ensure the required ventilation volume by the pressure rise (ΔP_r) generated by the jet effect of the Jet fan to supplement the traffic ventilation capability (ΔP_T) and the natural ventilation capability (ΔP_{MT}).
In case that the direction of ventilation wind in the tunnel, the direction of the jet flowing, the direction of the vehicle traveling, and the direction of natural ventilation are as shown in the figure below, the following relational formula holds.



Source: Road tunnel (Ventilation) technical standard and explanation; Japan Road Association 2010.

Figure 7-10: Model diagram for Jet-fan type ventilation pressure

$$\Delta P_r + \Delta P_{MT} = \Delta P_t + \Delta P_j$$

ΔP_r : Ventilation resistance
ΔP_{MT} : Natural ventilation capability
ΔP_t : Traffic ventilation capability
ΔP_j : The sum of pressure rise by Jet-fans

(1) Pressure rise by Jet-fans (ΔP_j)

The pressure rise by Jet-fan (ΔP_j) is obtained from Bernoulli's theorem and momentum equation as follows. (Meidinger equation)

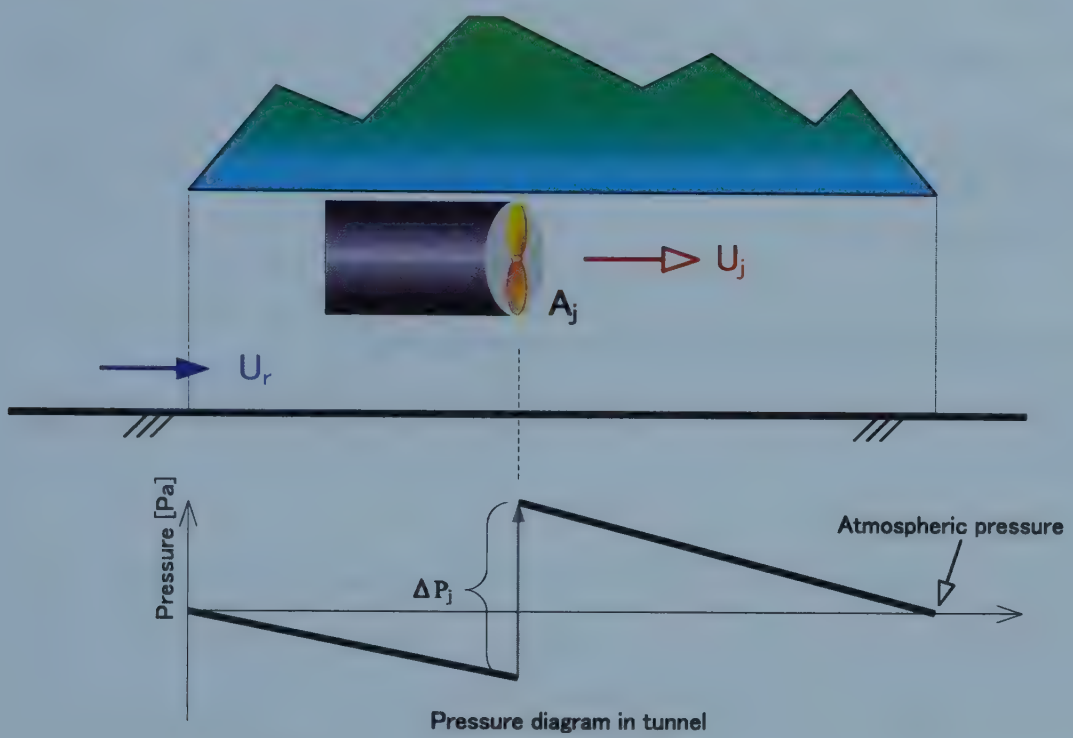
$$\Delta P_j = \frac{\rho}{2} \cdot U_j^2 \cdot \phi \cdot \frac{1-\psi}{(1-\phi)^2} \cdot (2 - 3\phi + \phi\psi)$$

φ : Cross-section area ratio = A_j / A_r
ψ : Wind speed ratio = U_r / U_j
A_j : Area of jet fan outlet [m^2]
U_j : Blowing speed from jet fan outlet [m/s^2]

Normally, the cross-section area ratio (φ) of the above equation would be less than 1 (φ << 1). Therefore, an approximation formula following can be used instead of the above regular equation.

$$\Delta P_j = \frac{\rho}{2} \cdot U_j^2 \cdot 2\phi \cdot (1 - \psi)$$

However, it should not desirable that this equation be used for the calculation of the number of Jet-fan required. Because the accuracy of this approximation formula is lower than that regular equation.



Source: JICA Project Team.

Figure 7-11: Model diagram for the pressure rise by Jet-fan (ΔP_j)

Moreover, the pressure rise by Jet-fan (ΔP_j) can be calculated by the number of Jet-fans installed (i) and the pressure rise constant (K_j) in the road when Jet-fans are installed in the following formula as well.

$$\Delta P_j = K_j \cdot \frac{\rho}{2} \cdot U_j^2 \cdot 2\phi \cdot (1 - \psi) \cdot i$$

K_j : Pressure rise constant in Jet-fan
i : Number of installed Jet-fans per a position [units]
The pressure rise constant (K_j) should apply the values in the following table according to the design of roadway wind speed.

Table 7-8: Pressure rise constant (K_j)

Roadway wind speed [m/s]	Installation interval [m]	Pressure rise constant (Kj)
less than 4	100	0.99
4 to 8	120	0.92
8 to 12	140	0.90

Source: Seventh collection, Design Manual of Japanese Expressway 2018

(2) The calculation for the number of Jet-fans

The pressure rise (ΔP_j) for generating the required ventilation volume shall be calculated as the Jet-fan ventilation capability to be used. And, the number of Jet-fans to be operated is derived from the calculation formula of the traffic ventilation wind (U_r) and capability (ΔP_τ) that satisfies the required ventilation volume (Q_{rq}) and the pressure rise (ΔP_j) per jet fan.
Thus, the required number of Jet-fan shall be calculated from the following formulas.

$$i = \frac{\Delta P_r \pm \Delta P_{MT} \pm \Delta P_t}{\Delta P_j}$$

i : Number of installed Jet-fans per a position [units]
ΔP_r : Ventilation resistance
ΔP_{MT} : Natural ventilation capability
ΔP_t : Traffic ventilation capability
ΔP_j : The sum of pressure rise by Jet-fans

(3) The calculation for the number of Jet-fans in one way tunnel

The required number of Jet-fan for one way tunnel should be calculated above formula (ΔP_j) and the following formulas.

$$\Delta P_r = (1+\zeta_e+\lambda \cdot \frac{L}{D_r}) \cdot \frac{\rho}{2} \cdot U_r^2$$

$$\Delta P_{MT} = (1+\zeta_e+\lambda \cdot \frac{L}{D_r}) \cdot \frac{\rho}{2} \cdot U_n^2$$

$$\Delta P_t = \frac{A_m}{A_r} \cdot \frac{\rho}{2} \cdot \frac{N \cdot L}{1000\ V_t} \cdot (V_{t+} -- U_r)^2$$

$$Z = \frac{\Delta P_r + \Delta P_{MT} -- \Delta P_t}{\Delta P_j}$$

Z : Number of installed Jet-fans [units]
ΔP_r : Ventilation resistance
ΔP_{MT} : Natural ventilation capability
ΔP_t : Traffic ventilation capability
ΔP_j : The sum of pressure rise by Jet-fans

(4) The calculation for the number of Jet-fans in two way tunnel

In case that the upward and downward traffic volume ratios are 50:50 respectively, the required number of Jet-fan shall be calculated from the following formulas.

$$\Delta P_r = (1+\zeta_e+\lambda \cdot \frac{L}{D_r}) \cdot \frac{\rho}{2} \cdot U_r^2$$

$$\Delta P_t = \frac{A_m}{A_r} \cdot \frac{\rho}{2} \cdot \frac{N_+ \cdot L}{1000\ V_t} \cdot (V_t -- U_r)^2 - \frac{A_m}{A_r} \cdot \frac{\rho}{2} \cdot \frac{N_- \cdot L}{1000\ V_t} \cdot (V_t + U_r)^2$$

In case that the upward and downward traffic volume ratios are 50:50 ($N_+ : N_- = 50:50$)

$$\Delta P_t = -\frac{A_m}{A_r} \cdot \frac{\rho}{2} \cdot 4 \cdot V_t \cdot U_r$$

$$n = \frac{N \cdot L}{1000\, V_t} \cdot \frac{1}{2}$$

Reverse pressure in the following formula is required when the natural wind (U_n) is opposite direction to direction of roadway wind

$$\Delta P_{MT} = (1 + \zeta_e + \lambda \cdot \frac{L}{D_r}) \cdot \frac{\rho}{2} \cdot U_n^2$$

Thus, the required number of Jet-fan shall be calculated from the following formulas.

$$\Delta P_j = K_j \cdot \frac{\rho}{2} \cdot U_j^2 \cdot 2\phi \cdot (1 - \psi)$$

$$Z = \frac{\Delta P_r + \Delta P_{MT} - \Delta P_t}{\Delta P_j}$$

(5) Round processing of the number of Jet-fans

The number of installed Jet-fans calculated by the above formulas would be rounding up. The processing of this rounding up can be a range to the caliber of one rank lower than the caliber calculated. And, considering maintenance, the minimum number of Jet-fan must be 2.

(6) Selection of Jet-fan shape

The diameter of the Jet-fan must be selected so that it can be arranged within the range defined by the construction limit.

(7) Layout

The Jet-fans should be installed with an appropriate layout so that their capabilities can be demonstrated to the utmost and maintenance and management expenses can be optimized also.

(8) Installation interval and distance from mine mouth of the tunnel for Layout

Installation interval of Jet-fans and separation from the tunnel mouth part are ensured in accordance with the following table.

Table 7-9: Installation interval of Jet-fans

Roadway wind speed [m/s]	Installation interval & Distance from mine mouth of tunnel [m]
less than 4	100
4 to 8	120
8 to 12	140

Source: Seventh collection, Design Manual of Japanese Expressway 2018

(9) Distance from emergency parking zone for Layout

Jet-fan should install with ensuring a separation distance of 120 [m] or more between the jet-fan discharge side and the downstream side end portion of the emergency parking zone. Also, the jet fan suction side can be installed at the downstream end of the emergency parking zone.

(10) Other consideration for Layout

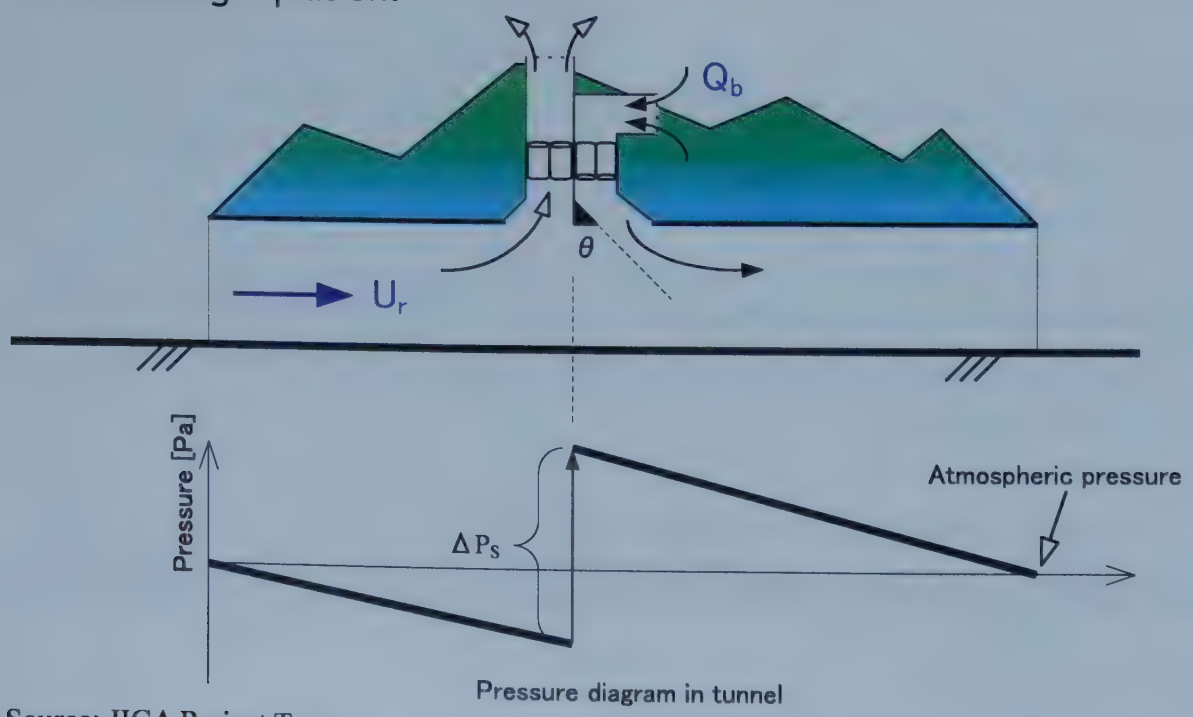
(a) The minimum installation number shall be two, and two units shall be installed in parallel on one cross-section in principle.
(b) Installation of Jet-fan should be close as possible to the electrical room. However, the installation of Jet-fan for smoke exhaust should be close to mine-mouth of the tunnel on the entrance-side. And, Jet-fans used for slowing the wind speed among their jet-fans should be located in consideration of stopping Jet-fans within about 500 [m] from the fire spot.
(c) The shape and mounting position of Jet-fan should be determined in consideration of the maintenance work which includes the lane regulation of the road.
(d) The separation distance between Jet-fan and the ceiling wall surface should be about 200 [mm].
(e) The separation distance between the Jet-fan and the end face of the wall shall be 200 [mm] or more.

(11) Fire protection measures for Jet-fans of smoke exhaust

(a) The cable for Jet-fans should be wiring within the duct in the tunnel inspection gallery.
(b) The wiring cable from Hand-hole to Jet-fans should be the fire-protected cable, and it should be wiring in the steel pipes.

7.14.4 Air supplying nozzle type of mechanical ventilation

The air supplying nozzle feature is to obtain the pressure rise (ΔP_s) by a jet of comparatively large air volume. And in the present situation, it is used in an air supply and exhaust system and a dust collector. The boosting force by this method is shown by the following equation.



Source: JICA Project Team.

Figure 7-12: Model diagram for the pressure rise by Air supplying nozzle (ΔP_s)

$$\Delta P_S = 2 \cdot \left(\frac{Q_b}{Q_r}\right) \cdot \left\{\left(\frac{Ub \cdot \cos\theta}{Ur}\right) - 2 + \left(\frac{Q_b}{Q_r}\right)\right\} \cdot \frac{\rho}{2} \cdot U_r^2$$

ΔP_S : Pressure rise by Air supplying nozzle
Q_r : Traffic ventilation volume [m^3] ($Q_r = U_r \times A_r$)
U_r : Roadway wind speed [m/s]
A_r : Roadway cross-section area [m^2]
Q_b : Air supplying nozzle ventilation volume [m^3]
U_b : Air supplying wind speed [m/s]
θ : Angle of Air supplying nozzle [m^2]

7.14.5 The upper limit of the roadway wind speed

The upper limit of the roadway wind speed in the tunnel must be set to 12 [m/s] in the case of one-way traffic and 8 [m/s] in the case of two-way traffic.

CHAPTER 8 Tunnel Lighting

8.1 Requirement

Road tunnels must be made so that the vehicles pass smoothly, speedily, and comfortably. As one of the obstacles to these features is that, sunlight does not reach inside the tunnel so that it is usually dark, making it difficult for the vehicle to pass smoothly, speedy, and comfortably.

Unless there are lights inside the tunnel, though vehicles would move with their own lights on, the speed of traffic would get reduced. This is because inside the tunnel is dark so that the driver must be driving with taking care until the end of the tunnel. This will also cause congestion and obstruction for smooth, efficient, and speedy movement of road traffic. Thus, covering the road in the tunnel with a bright environment is required to create an environment where the vehicle can run smoothly throughout the tunnel.

The requirement for this significant objective mentioned is influenced by several critical factors which determine visibility. These conditions are eminently variable and involve characteristics of the driver, including ability, age, and personal habits, the physical conditions of road and tunnel, the access road to the tunnel, the length of tunnel and road, atmospheric conditions, traffic density, traffic volume and speed, type of vehicles and regional character.

Moreover, the daytime and the nighttime lighting requirements are quite different because the visual perception of the human eye is greatly different when traveling on a road illuminated with a bright sun and when traveling on a road illuminated with headlights after the sun has settled. Therefore the minimum daytime and nighttime lighting requirement is to ensure visibility conditions so that the user may travel equally well by the daytime or by the nighttime at a given design speed. It is also true that the human eye takes a longer time in adaptation from light to dark than from dark to light. In other words, the design of lighting at the entrance zone of the tunnel should be given more attention than at the exit zone of the tunnel.

8.2 Criteria

Criteria for tunnel lightings should be considered by the environment around the tunnel. The criteria for installing lighting in the tunnel shall be in accordance with the guidelines indicated in 'IRC:SP: 91-2010' basically, and the special mention for the tunnel lighting on the highway in mountainous regions is mainly noted in this chapter.

In addition, the theoretical methods will be referred to with the international standard which is "CIE 88 2004 2nd edition GUIDE FOR THE LIGHTING OF ROAD TUNNELS AND UNDERPASSES" what 'IRC:SP: 91-2010' is based upon.

8.2.1 Definition of Quality for Lighting

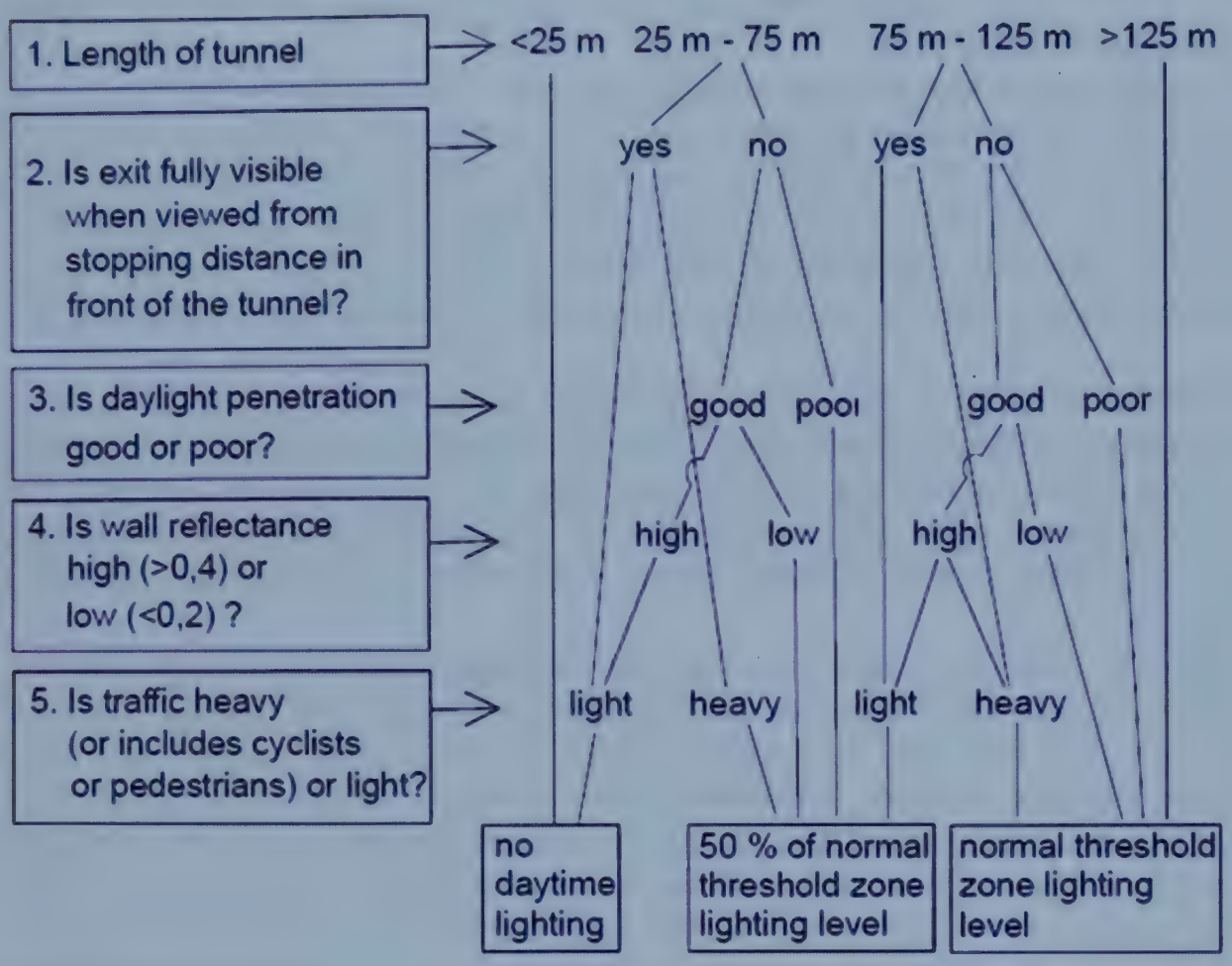
The photometric characteristics of the lighting installation of a tunnel which define the quality of the lighting system are as follows:

- the luminance level of the road and of the lower part of tunnel walls;
- the uniformity and distribution of the luminance of the road surface and of the walls;

- the limitation of glare produced by the light sources;
- the limitation of the flicker effect;
- the level of visibility of possible obstacles;
- the visual guidance.

8.2.2 The daytime lighting

The daytime lighting in the tunnel should be installed according to the figure below. For the tunnel lengths up to 75 [m] where no daytime lighting is recommended, it is to be noted that at least one hour before sunset and one hour after sunrise a lighting level equal to the recommended values for the interior zone of the long tunnel should be achieved (see luminance in the interior zone). At night only the recommended value for the nighttime lighting is required.



Source: IRC:SP: 91-2010.

Figure 8-1: The daytime lighting of tunnels for different tunnel lengths

8.3 Plan

The road tunnel consists of a tunnel and the roads connected before and after the tunnel. And, the road tunnel should become that these are united as a road. Thus, the road tunnel must function as a road.
Therefore, the tunnel lighting facility should make planned as a part of the road, not thinking only about the tunnel. Also, the tunnel lighting facilities is required to make a plan as a whole tunnel construction so that it is also related to the tunnel shapes and structures.

8.4 Investigation

The investigation which is consisted of the tunnel lighting facility, the tunnel shape / structure, traffic conditions, environment around tunnel portals, situation of related facilities, maintenance management should be required for planning and designing of tunnel lighting facility.

(1) Tunnel shape/structure

The investigation of tunnel shape/structure is required for the composition of tunnel lighting, selection of lighting equipment, mounting of lighting equipment, and calculation of utilization factor. Therefore, this investigation will carry out the following.

- The tunnel extension, sectional shape, width configuration, construction limit, planar and longitudinal linear, finish and reflectance of road surface/wall surface/ceiling surface, the tunnel wall painting (interior), the tunnel wall surface structure, the light reflection characteristic of tunnel wall surface

(2) Traffic conditions

The investigation of traffic conditions is required for examination of lighting method and calculation for the luminance of road surface and light loss factor. Therefore, this investigation will carry out the following.

- Design traveling speed, traffic volume, traffic method, and road classification

(3) The environment around the tunnel portals

The investigation of the environment around the tunnel portals is required for the calculation of luminance for entrance/exit parts and the requirement for lighting on connecting road. Therefore, this investigation will carry out the following.

- The azimuth of tunnel portals, topography/features in the vicinity of tunnel portals, linearity of the connecting road (planar linear and longitudinal gradient), weather conditions

(4) The situation of related facilities

The investigation of the situation of related facilities which are consisted of ventilation facilities and the emergency facility is required as follows, the setting precondition of size, the amount, the method, the arrangement, the arraignment, the cabling, and the operation of tunnel lighting.

(5) Maintenance management

The investigation of maintenance management is required for the traffic condition, operation, and controlling of the equivalent tunnel and near tunnel so that tunnel operation is carried out effectiveness and economically.

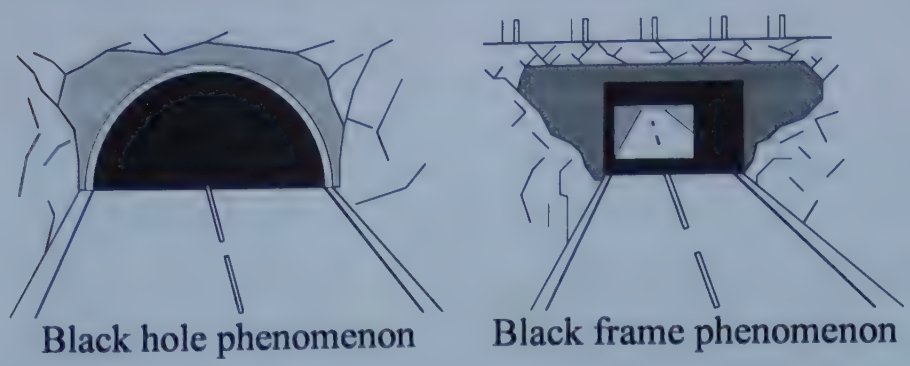
8.5 Visual phenomena

In the daytime, the driver who is driving on the road will experience several visual phenomena when traveling through the tunnel. These visual phenomena seen in each section of the road tunnel are explained below.

8.5.1 In case of being traveling toward the tunnel entrance

(1) Black hole phenomenon and black frame phenomenon

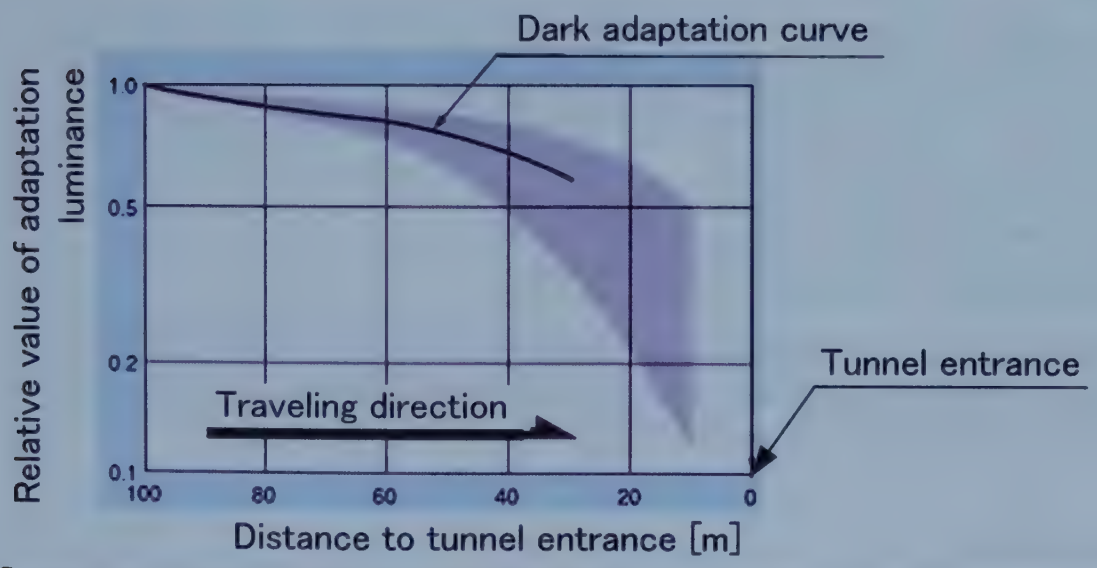
In a tunnel where the tunnel exit cannot be seen from the tunnel entrance, the driver feels that this tunnel looks like a dark hole when the driver is going toward the tunnel entrance. This is called the black hole phenomenon. On the other hand, the tunnel entrance looks like a dark frame in a short tunnel where the light at the tunnel exit can be seen. This is called a black frame phenomenon. In either case, the driver can lose his sight of other cars, structures, obstacles, and the situation inside the tunnel.



Source: JICA Project Team.

Figure 8-2: Black hole phenomenon and black frame phenomenon

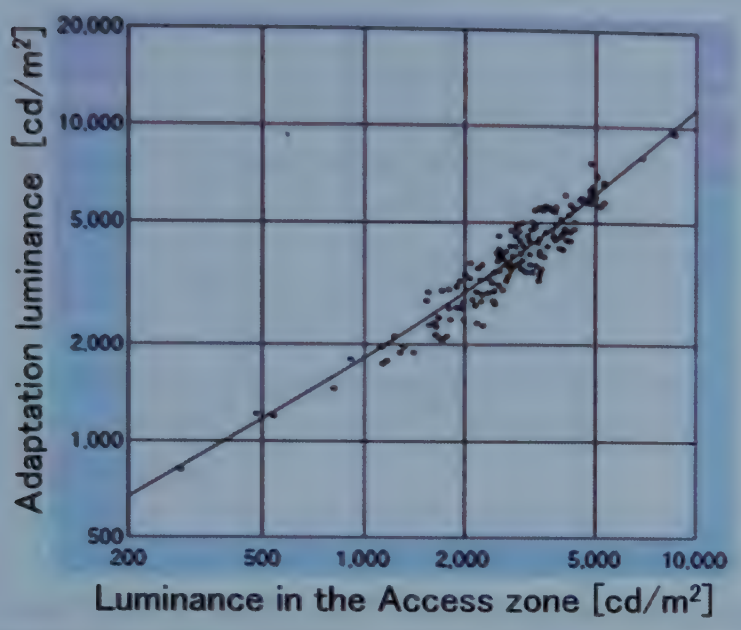
Generally, in order for a driver to drive a car safely, the driver must ensure a distance (safety stop distance (SD)) necessary for the driver to sufficiently discriminate the road surface condition in front of the vehicle. Also, even if there was enough distance from the preceding car, the driver might feel danger. Hence, the driver is driving to take care of such distance and around situations until the end of his driving.
According to the above reason, the driver can feel danger if the situation of the road surface in front has become not to be able to ensure the safety stop distance by black hole phenomenon or black frame phenomenon. Therefore, proper lighting is required at the tunnel entrance part.
Experimental result of the black hole phenomenon has been reported from the CIE. The following figure is the relation between the distance and the adaptation luminance of the driver's eyes when it is approaching the tunnel entrance confirmed by the experiment of the black hole phenomenon. Dark adaptation curve means making a relation between the Relative Value of the adaptation luminance and Distance to tunnel entrance when the driver's eye is approaching the tunnel entrance.



Source: Road tunnel (Lighting) technical standard and explanation; Japan Road Association 2009.

Figure 8-3: Adaptation luminance of driver's eyes before entering the tunnel

According to the above figure, as the car approaches the entrance of the tunnel, the driver's eyes gradually decrease the adaptation luminance.
This reason is that the driver's field of view is gradually occupied by the dark part of the tunnel entrance so that the scattered light in the driver's eyeball decreases then darkness occupies the driver's eyeball.
As a result, the adaptation luminance gradually decreases due to the influence of the scattered light in the driver's eyeball.



Source: Road tunnel (Lighting) technical standard and explanation; Japan Road Association 2009.

Figure 8-4: Relationship between the luminance and adaptation luminance in the access-zone

Moreover, CIE conducted experiments on the relationship between the adaptation luminance in the driver's eyes and the luminance in the access zone.
According to Figure 8-4, the experimental results show that as the luminance in the access zone (luminance outside the tunnel) increases, the adaptation luminance of the driver's eyes also increases. This means that the adaptation luminance of the driver's eye is proportional to the luminance in the access zone.
Therefore, the tunnel entrance illumination curve can be set by using luminance in the access zone which is easy to calculate and measure instead of adaptation luminance.

(2) Veiling phenomenon

The exhaust fumes get inside the tunnel when the tunnel is long, or the traffic volume increases. This accumulated exhaust fumes will gradually flow out of the tunnel entrance.
The exhaust fume will shine brightly when the direct light from the sun Illuminates this exhaust fumes that flow out from the tunnel entrance. These brightly shining exhaust fumes occur between the car traveling towards the tunnel entrance and the tunnel entrance, making inside the tunnel invisible.
This phenomenon is called the Veiling phenomenon. This can be reduced by installing a forced-air ventilation facility and the louver for light shielding of about 10 [m] so that sunlight does not directly illuminate the exhaust fumes flowing out of the tunnel.

8.5.2 In case of just being entered in the tunnel

(1) Blackout phenomenon

Generally, immediately after entering the dark space in the tunnel from the high brightness outdoor, it seems extremely dark inside the tunnel. This is the same phenomenon as the lighting in the room just after the power failure.
The reason for this is that driver's eyes are adapted to the high luminance of the outdoor luminance and take time to adapt to the luminance (dark space) inside the tunnel. This phenomenon is called the Blackout phenomenon.
The driver feels dangerous that such a phenomenon also makes it impossible to understand the situation ahead.
In order to solve this phenomenon, proper lighting is required in the Tunnel entrance.

8.5.3 In case of being travel inside the tunnel

(1) Lowering of the visibility by the exhaust fumes

In case the exhaust fumes (especially soot) is incomplete in the tunnel ventilation, the exhaust fumes will stagnate in the tunnel as the traffic volume increases, the driver's visibility will decrease, and the drivers will feel scared and dangerous
A ventilation facility should be installed in the tunnel to secure the transmissivity of the tunnel.
Moreover, the design value of the road surface luminance in the tunnel is set considering the transmittance per 100 [m].

(2) Flicker phenomenon

In case the lighting fixtures are installed discontinuously, the driver feels discomfort due to these repeated light and dark which is called Flicker. The Flicker phenomenon also occurs due to the luminance ratio between bright and dark, and the time ratio between bright and dark. Experimental results on the Flicker phenomenon are shown below.

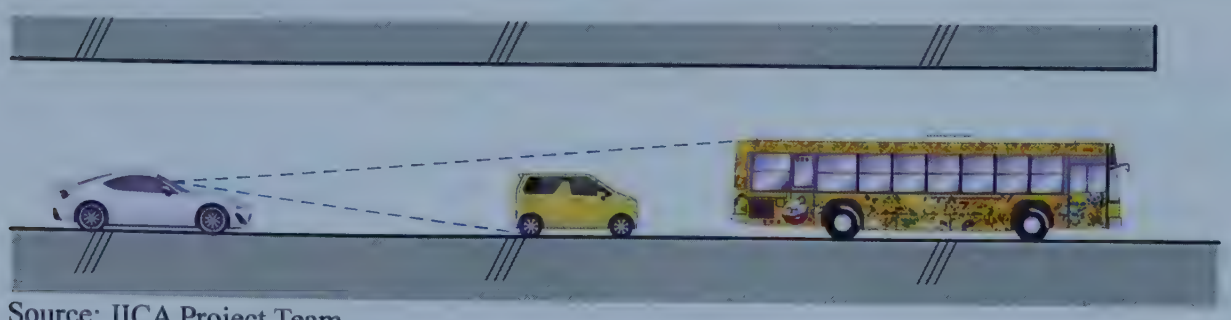
- The driver feels most uncomfortable when the time ratio bright to the dark range with 5 to 18 [Hz] as repetition frequency.
- However, even at the above frequencies, this discomfort will be reduced if the luminance ratio bright to dark is less than 10 [nos].
- In the time ratio brightness to darkness, it will be most unpleasant when brightening time in this cycle is 25 [%].
- The discomfort is reduced as this is greater or less than 25 [%].

Tunnel lighting design should not use such unpleasant frequency (time ratio between bright and dark) or luminance ratio between bright and dark.

8.5.4 In case of being travel toward the tunnel exit

(1) The decline in visibility near exit inside the tunnel (in the daytime)

During the daytime, the driver can see the outside field very bright from inside the tunnel when the car is traveling toward the tunnel exit. And this driver will see that this tunnel exit part seems a white hole. At this time, some obstruct near the tunnel exit and the preceding car is black silhouettes with the bright outdoor brightness as background, their identification is easy and there is no occurred particular problem for the driver's visibility.



Source: JICA Project Team.

Figure 8-5: Decline in visibility near the exit

All preceding vehicles will appear dark and these preceding vehicles will look like one car if there are two or more preceding vehicles between the host car and the tunnel exit according mentioned above. With this phenomenon, the driver will not be able to determine how many distances are necessary between the host car and the preceding vehicles. As a result,

the driver will not be able to get a safe stopping distance and some dangerous situations will arise.
By lighting in the Tunnel exit can be for the driver to identify the preceding vehicle properly and to be known exactly the traveling speed and distance of the preceding car. In order to prevent this phenomenon, lighting in the Tunnel exit should be required.

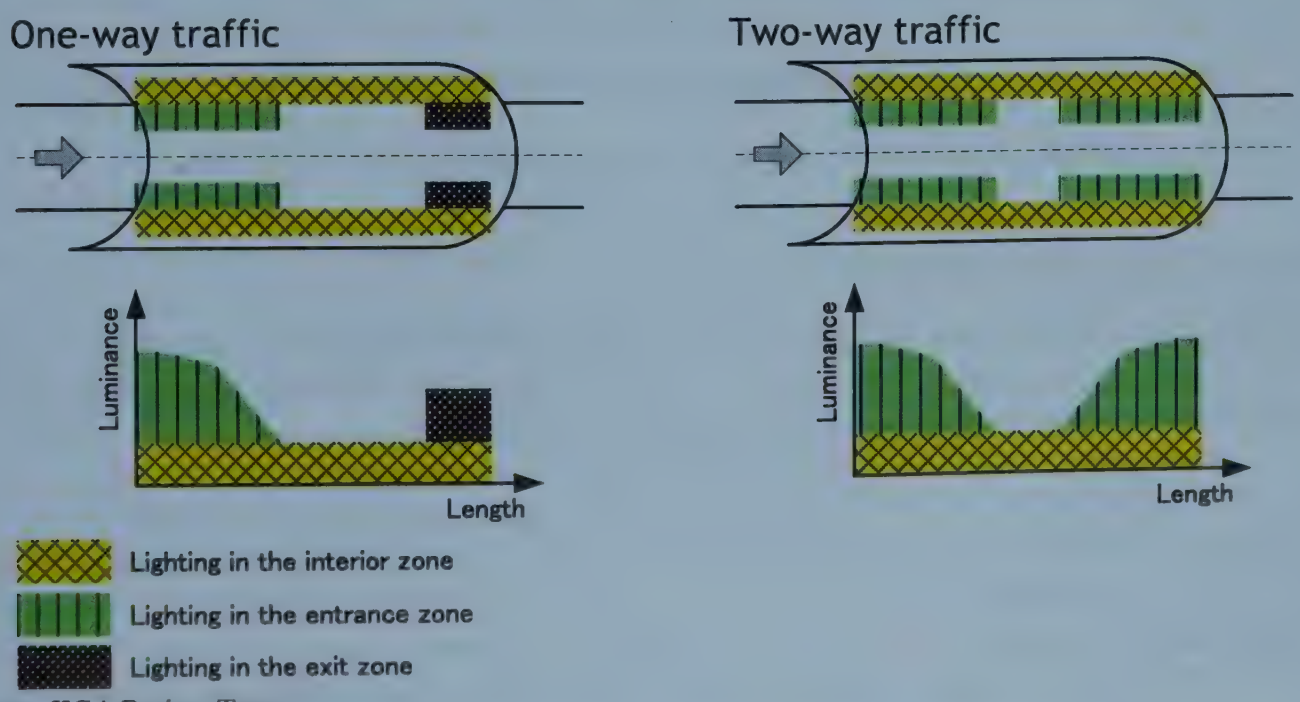
(2) The decline in visibility near the exit inside the tunnel (During the nighttime)

During the nighttime, in case there is lighting only inside the tunnel and there is no lighting on the road connected to the tunnel exit, or even if there is lighting on the connecting road, the driver feels extremely dark, the driver will be feeling the blackout phenomenon.
The driver feels extremely dangerous because it is also not known about some obstructs on the tunnel connecting road and the road shape when this phenomenon occurs.
For this purpose, it is necessary to install lighting on the connecting road of the exit so that the road surface is sufficiently bright. Or, take measures to recognize at least changes in road linearity.

8.6 Configuration of Tunnel lighting

The facility of lighting in the tunnel composed of the lighting in the interior zone, the lighting in the tunnel entrance, and the lighting in the exit zone mainly. And this configuration has to change in accordance with depending on the traffic system. The arrangement for these lightings is shown in the figure below.
In the case of Two-way traffic, the lighting in the tunnel entrance shall be arranged both tunnel entrances so that the lighting in the tunnel entrance can perform the lighting function of the lighting in the exit zone, also.
The lighting in the interior zone is the basic lighting of the tunnel lighting facility. In the facility of lighting in the tunnel, in addition to the lighting in the interior zone for securing the brightness throughout the tunnel, there is partial lighting for securing the brightness in the local part of the tunnel where is like the following part. These are particularly necessary illumination to ensure brightness beyond the basic lighting (the lighting in the interior zone).

- The tunnel entrance, the exit of the tunnel, the special structure part in the tunnel



Source: JICA Project Team.

Figure 8-6: Configuration of Tunnel lighting

8.6.1 Lighting in the interior zone

The lighting in the interior zone is for ensuring the brightness necessary so that the driver traveling in the tunnel views the obstacles and the preceding vehicle in the tunnel. Therefore, this lighting is installed over the entire length of the tunnel to ensure the brightness necessary throughout the tunnel. And, the lighting in the interior zone shall be designed to satisfy the road surface luminance, the uniformity ratio of illuminance, the glare restrictions, and the inductivity because of ensuring safety and comfortableness for the driver traveling in the tunnel. In addition, the wall luminance, the arrangement of the lights, and the flicker are also required to consider for ensuring safety and comfortableness for the driver traveling in the tunnel.

(1) Arrangement

Lighting in the interior zone is arranged at regular intervals over the entire length of the tunnel.

(2) Average road surface luminance

The road surface luminance of Lighting in the interior zone depends on the design traveling speed. The relationship between road surface luminance and design traveling speed is shown below table,

Table 8-1: Average road surface luminance for Lighting in the interior zone

Design traveling speed [Km/h]	Average road surface luminance [cd/m^2]
100	9.0
80	4.5
70	3.2
60	2.3
50	1.9
40 and less	1.5

Source: Road tunnel (Lighting) technical standard and explanation; Japan Road Association 2009.

This table from CIE shows the road surface luminance required for tunnels by design s traveling peed in case of the transmittance in the tunnel is equal greater than 50 [%] or per 100 [m].
Accordingly, road surface luminance can be reduced when the transmittance is high (traffic volume is small).
However, the road surface luminance cannot be set to less than 0.7 [cd/m^2].

(3) Tunnel wall surface luminance

The balance of total luminance should not care only the luminance of the road surface but also the luminance of the tunnel wall surface and the luminance of the tunnel ceiling because the tunnel is composed of roads and walls.
Particularly, since the tunnel wall surface is often the background of the preceding vehicle and/or obstacle, in order for the driver traveling in the tunnel to confirm the safety distance to the preceding vehicle, it is important to make clear the road and wall boundary. Thus, the tunnel wall surface luminance must be considered as a part of lighting in the interior zone sufficiently. In addition, the tunnel wall surface luminance sets a range from the road to the height 1 [m] because the obstruction is existing within 1 [m] height on the road.
In the case of installing a tunnel interior (such as an interior panel), the tunnel wall surface luminance becomes equivalent to the road surface luminance. However, when

the road surface is white-based painting and the road shoulder is 0.7 [m] or less, the tunnel wall surface luminance is 1.5 times the road surface luminance in order to improve the clarity of the road surface obstacle. Furthermore, when there is no installation of the tunnel interior, the tunnel wall surface luminance is 0.6 times the road surface luminance. Also, when the road surface is painted white and the wall surface luminance becomes equal to the road surface luminance, the boundary between the road surface and the wall surface will be unclear. In such a case, clarify the lane marking of the road and the rising part of the curb.

(4) Uniformity ratio of illuminance

The visibility inside the tunnel is influenced not only by road surface luminance but also by the uniformity ratio of illuminance. Therefore, reducing unevenness in road surface luminance is required.
The total uniformity ratio of illuminance should be more than 0.4 at least. Also, in order to alleviate visual discomfort, it is recommended that the lane axis uniformity ratio of illuminance should be more than 0.6.
The formulas for calculating the total uniformity ratio of illuminance are shown below.

- Total Uniformity ratio of illuminance (U_0) = L_{min} / L_r

L_{min} : Minimum of Luminance of other parts [cd/m^2]
L_r : Average of road surface luminance [cd/m^2]

OR

- Total Uniformity ratio of illuminance (U_0) = L_{min} (l) / L_{max} (l)

L_{min} (l) : Minimum of Luminance of other parts on the lane center line [cd/m^2]
L_{max} (l) : Maximum of Luminance of other parts on the lane center line [cd/m^2]

(5) Glare Restrictions

The visibility inside the tunnel is also affected by Glare. Therefore, Glare restriction is required. The relative threshold increment TI which digitizes Glare Restrictions must be less than 15 [%]. Also, according to the Average road surface luminance, calculate the relative threshold increase Tl from the following formulas.

In case of Lr <= 5 [cd/m^2]	In case of Lr > 5 [cd/m^2]
Tl = 65 x (L_v / $L_r^{0.8}$) [%]	Tl = 95 x (L_v / $L_r^{1.05}$) [%]

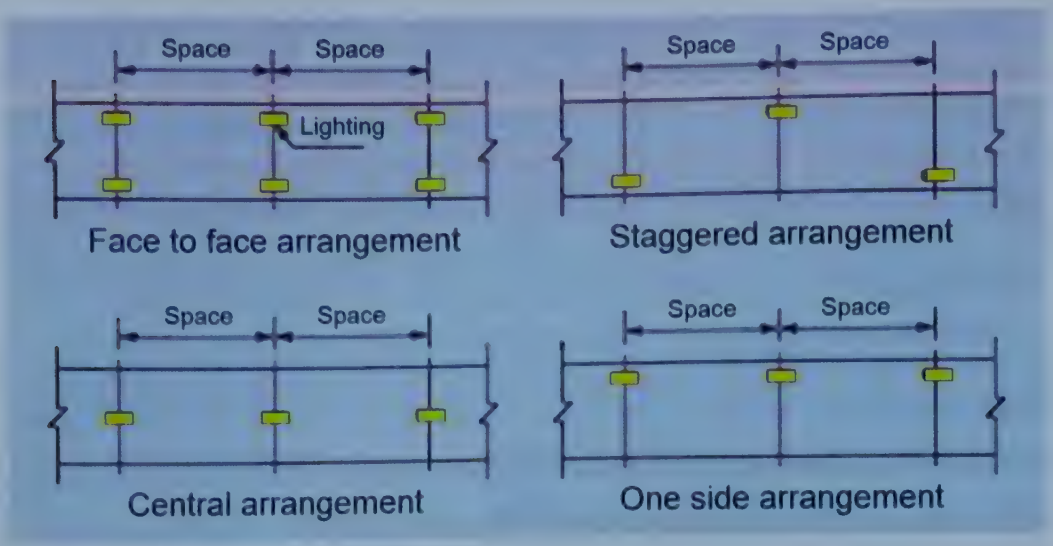
L_r : Average of road surface luminance [cd/m^2]
L_v : Equivalent veiling luminance for the lighting [cd/m^2]

(6) Inductivity

Inductivity consists of visual inductive and optical inductive. In case that the average of road surface luminance and the total uniformity ratio of illuminance is satisfied the standard values, the effect of visual inductive will be maximized. Also, by considering the height, arrangement, spacing of the lighting, it is possible to obtain the effect of optical inductive as well. The inductivity must be required to ensure itself by the tunnel lighting.

(7) Arrangement of lighting

There are four types of lighting arrangements for tunnel lighting, as shown face to face arrangement, staggered arrangement, central arrangement, and one side arrangement.



Source: JICA Project Team.

Figure 8-7: Arrangement of Lighting

The face-to-face arrangement has a high uniformity ratio of illuminance and inductivity; it is adopted for tunnel lighting requires highly average road surface luminance.
The staggered arrangement can be adopted for tunnel lighting with low traffic so that the average road surface luminance is reduced. However, as the installation interval of the lighting increases, the uniformity ratio of illuminance in the wall decreases, and the comfortableness is impaired.
The central arrangement is adopted for tunnel lighting with a small tunnel cross-section with a small traffic volume. Since the lightings are arranged on the roadway, it is necessary to consider safety such as the prevention of falling of the lighting fixtures.
In the one-sided arrangement, since the distance between the lightings is shortened, the inductivity becomes high in the case of straight tunnels. However, in the case of a curved tunnel, this lighting must be installed on the outer edge of the curve.

(8) Flicker

The following table shows the installation intervals of tunnel lighting where the Flicker are occurred according to the above mention. The lighting arrangement needs to avoid the installation interval in the table below.
However, the discomfort caused by Flicker affects not only the time ratio (frequency) between bright and dark caused by the lightings interval but also the luminance ratio between bright and dark. If this luminance ratio between bright and dark is less than 10, it is not necessary to avoid the lighting installation interval shown in the table below.

Table 8-2: Flicker and lighting installation interval

Design traveling speed [Km/h]	Lighting installation interval [m]
100	1.5 to 5.6
80	1.2 to 4.4
70	1.1 to 3.9
60	0.9 to 3.3
50	0.8 to 2.8
40	0.6 to 2.2

Source: Road tunnel (Lighting) technical standard and explanation; Japan Road Association 2009.

(9) Light reduction

When the traffic volume decreases (such as the nighttime and the late night), the exhaust fumes concentration inside the tunnel decreases. And the transmittance in the

tunnel will be better. In case, the average road surface luminance of the interior zone can apply the light reduction.
The method of light reduction has step-less light reduction and two steps light reduction that light reduction according to the decrease in traffic volume. Basically, two steps light reduction should be adopted. This reduction applies half of Primary light reduction and a quarter of Secondary light reduction. However, the average road surface luminance cannot be set to less than 0.7 [cd/m^2].

8.6.2 Lighting in the tunnel entrance

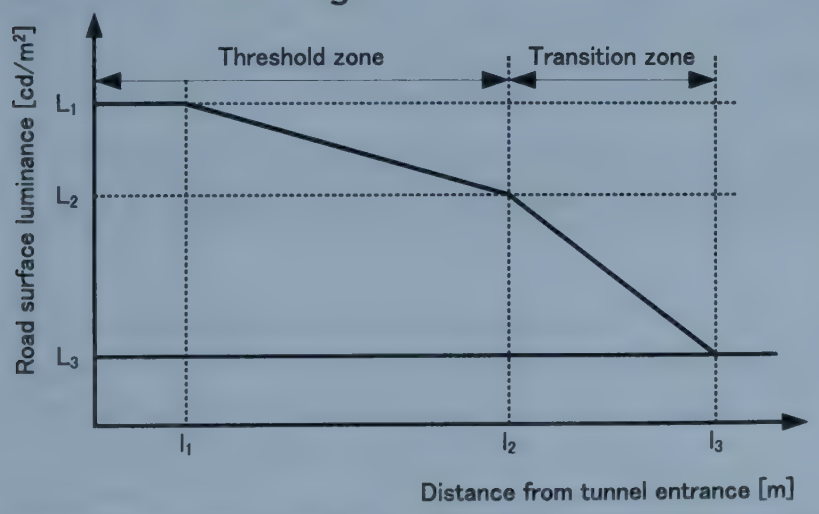
In the daytime, the visibility of the driver decreases due to the illuminance difference inside and outside the tunnel near the Tunnel entrance. In order to prevent it, the lighting in the interior zone at the tunnel entrance is required to enhance illuminance.
Therefore, Lighting for the Tunnel entrance is required to enhance the illuminance in the tunnel entrance section.
Normally, in the section from the tunnel entrance to 10 [m], the luminance of the road surface inside the tunnel is ensured by the injection of natural sunlight.
Therefore, In the case of a tunnel with a tunnel length of 25 [m] or more, or in the case of a tunnel where the tunnel exit cannot be seen from the tunnel entrance at a safe distance point, lighting for Tunnel entrance must be required. Lighting for Tunnel entrance consists of a Threshold zone (two levels) and a Transition zone.

(1) Arrangement

Arrangement of lighting for the Tunnel entrance should follow the arrangement of Lighting in the interior zone. This arrangement is started from the point of 10 [m] from the tunnel entrance because the luminance of the road surface in the tunnel entrance is ensured by the injection of natural sunlight. In the case of a curved tunnel entrance, this lighting must be installed on the outer edge of the curve.

(2) Road surface luminance of Lighting for the Tunnel entrance

Road surface luminance of Lighting for Tunnel entrance consists of three zones depending upon tunnel length. These zones of Lighting for the Tunnel entrance are shown the below figure.



Source: Road tunnel (Lighting) technical standard and explanation; Japan Road Association 2009.

Figure 8-8: Lighting for the Tunnel entrance

The luminance is standardized by CIE. These standard values are shown in the table below. The road surface luminance in the table below is the standard value when the luminance in the access zone is 3,300 [cd/m^2].

Table 8-3: Road luminance in Tunnel entrance

Design traveling speed [Km/h]	Road surface luminance [cd/m^2]			Length of tunnel entrance parts [m]			
	L_1	L_2	L_3	ℓ_1	ℓ_2	ℓ_3	ℓ_4
100	95	47	9.0	55	150	135	340
80	83	46	4.5	40	100	150	290
70	70	40	3.2	30	80	140	250
60	58	35	2.3	25	65	130	220
50	41	26	1.9	20	50	105	175
40	29	20	1.5	15	30	85	130

Source: Road tunnel (Lighting) technical standard and explanation; Japan Road Association 2009.

Note;

(a) L_1 is road surface luminance for the beginning of the primary threshold zone, L_2 is road surface luminance for the end of secondary threshold zone, L_3 is road surface luminance for the end of transition zone which is the beginning of the interior zone, l_1 is the length of primary threshold zone, l_2 is the length of secondary threshold zone, l_3 is the length of the transition zone, l_4 is the total length of entrance zone of tunnel (= $l_1 + l_2 + l_3$).

(b) In case that Luminance in the access zone is not 3,300 [cd/m^2], L_1 and L_2 is set up to proportion with the luminance in the access zone 3,300 [cd/m2]. Also, l_3 is calculated next formula; $l_3 = (\log_{10}L_2 - \log_{10}L_3) \times (V / 0.55)$ [m]

(c) In the case of oncoming traffic, the Road surface luminance of both tunnel entrances is conducted from the above table.

(d) In the case of design traveling speed is lower than 80 [km/h], Comprehensively considering the situation of roads and traffic, if road safety is not hindered, Road surface luminance can be reduced with the lower limit of half of the numerical value shown in Table 8-3 as the lower limit.

(e) In the case of the counter beam lighting method, Luminance in the primary threshold zone can be reduced by 20 [%]. However, if the luminance in the primary threshold zone could be half of L_1 of 60 [Km/h] as design traveling speed on table 2-3, Luminance in the primary threshold zone must not be reduced to 20 [%]. Because this effect is reduced by the influence of natural light.

(f) Luminance in the secondary threshold zone and Luminance in the Transition zone follow Table 8-3. Incidentally, the relative threshold increase shall be less than 15 [%].

(g) In the case of the pro-beam lighting method, Road surface luminance is shown in Table 8-3 and the vertical illuminance (road surface height 0.7 [m]) shown in Table 8-4 shall be secured.

Table 8-4: Required vertical surface luminance

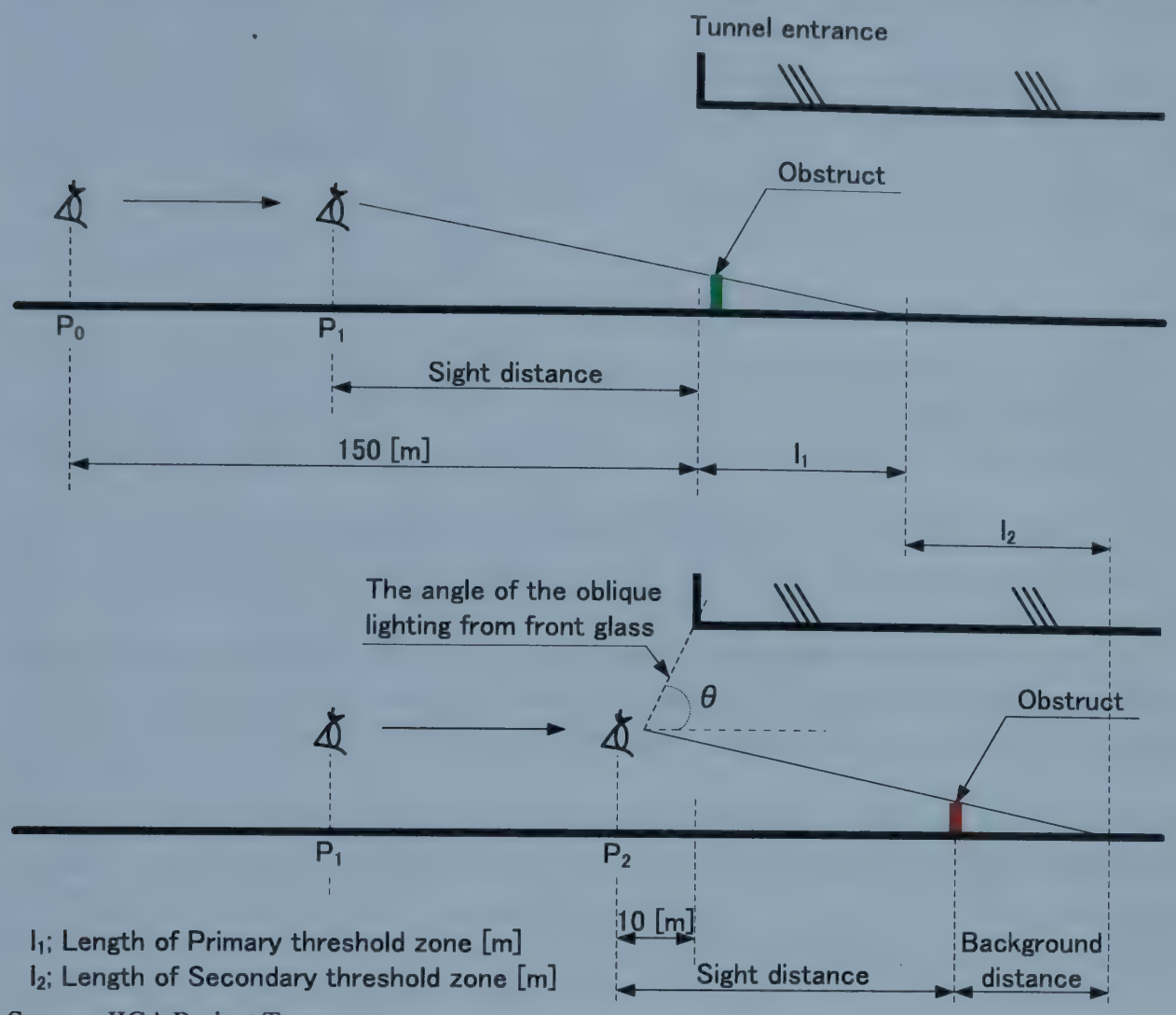
Design traveling speed [Km/h]	60	70	80	100
Vertical surface luminance [lx]	1,200	1,400	1,600	1,900
Length of each zone [m]	60	70	80	100

Source: Road tunnel (Lighting) technical standard and explanation; Japan Road Association 2009.

(h) Lighting for the tunnel entrance can be set up till 40 [m] from the tunnel exit if the lighting for the tunnel entrance enters within 40 [m] from the tunnel exit.

(i) At the tunnel entrance, it is recommended to plant trees and a sun-shaded louver to alleviate the sudden change of light and darkness due to natural light.

(3) Road luminance and Length in the primary threshold zone



Source: JICA Project Team.

Figure 8-9: Length of the Threshold zone

As shown in Figure 8-8, the purpose of the lighting of the primary threshold zone is that the driver can identify some obstruct near the entrance of the tunnel from P_1 point away from the tunnel entrance by the sight distance. Luminance in the primary threshold zone should be set to ensure constant visibility according to the adaptation luminance of the driver's eyes.

According to CIE, the adaptive luminance of the driver's eyes running towards the tunnel is influenced by more the equivalence veiling luminance than the fovea luminance of the driver in the section before the tunnel entrance in the daytime. The fovea adaptation is the luminance of the part drawn at an opening 2 degrees angle from the line of sight center.

Also, the adaptation luminance of the driver's eyes changes as shown in Figure 8-3. Figure 8-3 is a normalized curve with Adaptation luminance at the point 100 m before the tunnel entrance.

As shown in Figure 8-3, the adaptation luminance of the driver's eyes gradually decreases as approaches the tunnel entrance. The reason is that the driver doesn't only gaze at the tunnel but the influence of the high luminance portion such as the sky also became small. Also, there is a relationship as shown in Figure 8-4 between the adaptation luminance of the driver's eye and the luminance in the access zone.

According to the above mentions, it is recommended that the road surface luminance and length in the primary threshold zone use the luminance in the access zone which can be calculated and measured more easily than the adaptation luminance of the driver's eye.

Also, the adaptation luminance of the driver's eyes is set up as the luminance in the primary threshold zone based upon the luminance in the access zone at the point of 150

[m] (Point P0at upper figure of Figure 8-9) in this guideline. This reason is that Luminance in the access zone becomes nearly constant at over 100[m] from the tunnel entrance, the luminance in the access zone is set up at a point of 150 [m] from the tunnel entrance.
As shown in Figure 8-8 <Lighting for Tunnel entrance>, the length of the Primary threshold zone is equal to the length of some obstructing background. Also, the Length of the primary threshold zone shall be decided by the size of some obstruct, the sight distance as design traveling speed, and the height of the driver's eyes.

(4) Road luminance and Length in the secondary threshold zone

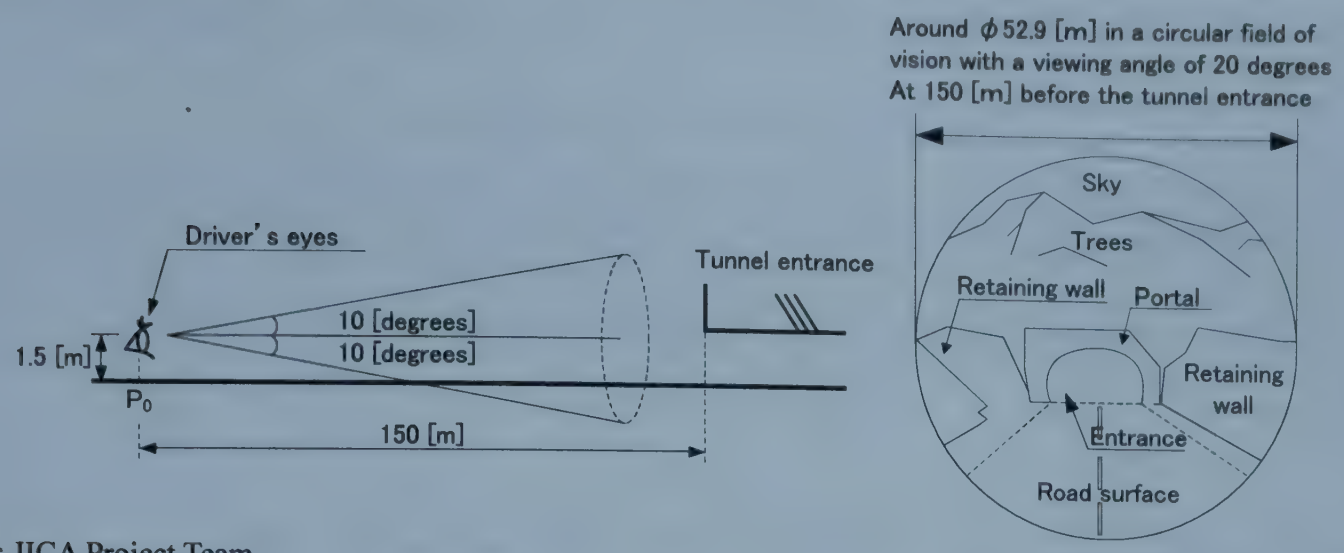
As shown in Figure 8-8, the purpose of the Lighting of the secondary threshold zone is that the driver can distinguish some obstruct in the deep tunnel from P_2 point. As shown in Figure 8-3, the Adaptation luminance of the driver's eye gradually decreases as approaches the tunnel.
Therefore, the road surface luminance required to identify obstacles can be lower than the road surface luminance required at point P_1. This continues from the entrance of the tunnel to P_2 at 10 [m].
The light-blocking angle from the automobile windshield at point P_2 is about 20 degrees with respect to the line of sight. At the P_2 point, the driver's field of view will occupy the entire dark part of the tunnel entrance. This is the same as visually entering the tunnel.
Therefore, the Length in the secondary threshold zone is calculated based on Point P_2 at the lower figure of Figure 8-8 (10 [m] from the entrance of the tunnel), and it is a value obtained by subtracting 10 [m] from the viewing distance. Also, Luminance in the secondary threshold zone is the average of adaptation luminance shown in Figure 8-3.

(5) Road surface luminance and Length in the Transition zone

Adaptation luminance of the driver's eyes begins to drop sharply when approaching the tunnel entrance from point P_2 in Figure 8-8. The road surface luminance is necessary to compensate for this decrease follows this curve for adaptation to luminosity.
Therefore, the road luminance in the Transition zone is calculated by it which is connected from the road surface luminance at end of the secondary threshold zone to the road surface luminance at beginning of the interior zone. As shown in Table 8-3, the length of the transition zone is obtained by graphs and formulas. The length of the transition zone is set in units of 5 [m].

(6) Luminance in the access zone

As described above, the luminance in the access zone is the average luminance within the circular field of view with a viewing angle of 20 [deg.] in the centered on the tunnel entrance when it is seeing the tunnel entrance from the road height of 1.5 [m] at the point P0 of 150 [m] from the tunnel entrance. The measured height (1.5 [m]) of the luminance in the access zone is the viewing angle height of the driver of a small car and a large car. Also, it must be set considering the azimuth of the entrance of the tunnel, the topography of the surroundings, and features.



Source: JICA Project Team.

Figure 8-10: Luminance in the access zone

There are three setting methods of luminance in the access zone, "A) Calculation", "B) Selection from Table", "and C) Measuring in the actual field".
Among the three methods, for the luminance in the access zone "C) Measuring in the actual field" method is the most desirable setting method after the tunnel entrance is completed.
However, the luminance in the access zone frequently needs to be set before tunnel portals and connecting roads are completed. Thus, "Measuring in the actual field" is less possible.
On the other hand, "B) Selection from Table" method is a useful method at a stage with less data on the information near the entrance of the tunnel so that is a convenient way to use as a design measure.
Compared with the above two methods, "A) Calculation" method is practical. It has been confirmed that the accuracy of the calculation result satisfies the required accuracy by CIE.
Three setting methods of luminance in the access zone will be described below.

(A) Method of Calculation;
Luminance in the access zone (L_{20}) is calculated by multiplying area ratios of the sky and plan metric feature occupied in a circular field of vision with a viewing angle of 20 degrees by the luminance of other parts in the table below and adding each term.

$$L_{20} = A_s \cdot L_S + A_r \cdot L_r + A_e \cdot L_e + A_h \cdot L_h \,[\mathrm{cd/m^2}]$$
$$(A_s + A_r + A_e + A_h = 1)$$

L_S : Luminance of Sky [cd/m²]
L_r : Luminance of Road surface [cd/m²]
L_e : Luminance of Portal area [cd/m²]
L_h : Luminance of sky from a tunnel inside [cd/m²]
A_h : Area ratio of the sky from a tunnel inside [m²]
A_s : Area ratio of the sky [m²]
A_r : Area ratio of road surface [m²]
A_e : Area ratio of portal [m²]

Generally, since the luminance of sky form tunnel inside shall be relatively low. Thus, the fourth term of the above equation will be zero on the calculation. And, the luminance in the access zone (L_{20}) calculated rounds to the tens place.

i) The luminance of other parts
The luminance of other parts is shown in the table below.

However, in the case of measuring the Luminance of other parts on-site, use the result.

Table 8-5: Luminance of other parts

Direction of entrance	Sky luminance(L_s) [cd/m²]	Road surface luminance(L_r) [cd/m²]	Luminance around tunnel entrance (L_e) [cd/m²]			
			Retaining wall	Tree	Building	Grass
North	13,000	4,000	2,000	1,500	2,000	2,000
East / West	8,000	3,500	2,000	1,500	3,000	2,000
South	7,000	3,000	3,000	2,000	4,000	2,000

Source: Road tunnel (Lighting) technical standard and explanation; Japan Road Association 2009.

Note;

- The azimuth of the tunnel entrance indicates the direction the entrance is facing. Also, the traffic direction is opposite.
- In the case of the northeast, northwest, south-east, and south-west where the direction of the tunnel entrance is not in the above table, the Luminance of other parts shall use the average value corresponding to each direction in the table above.
- The luminance of other parts in the above table is not a figure that considers snow accumulation.

ii) Area ratio

Area ratios of sky and plan metric feature occupied in a circular field of vision with a viewing angle of 20 degrees are calculated by two kinds of methods which are as follows.

- Using as build drawing; Draw a circle with a viewing angle of 20 degrees around the entrance of the tunnel, sketch the circumstances in the circle and obtain the area ratio.
- Using photography; At the point 150 [m] from the tunnel entrance (P_0), draw a circle with a viewing angle of 20 degrees centered around the tunnel entrance and sketch the circumstances in the circle to find the area ratio.

(B) Selection from Table;

In case there is any unknown situation near the tunnel entrance, the luminance in the access zone is calculated in the area ratio of the sky in the field of view (A_s) and the following table.

Table 8-6: Method of Selection from Table

Area ratio	$A_s < 0.05$	$0.05 \leqq A_s < 0.15$	$0.15 \leqq A_s < 0.25$	$0.25 \leqq A_s$
The luminance in the access zone [cd/m²]	2,500	3,300	4,200	5,000

Source: Road tunnel (Lighting) technical standard and explanation; Japan Road Association 2009.

Note;

A_s : Luminance in the access zone when area ratio of the sky in the field of view 20 [deg.].

(C) Measuring in the actual field;

The directly measuring method for the luminance in the access zone is most desirable than other methods. If the annual luminance in the access zone can be measured continuously, the cumulative appearance time-frequency of 95 [%] using these data is assumed to be luminance in the access zone (L_{20}).

8.6.3 Lighting in the exit zone

Generally, the natural light is injected inside to 40 [m] from the tunnel exit. Therefore, during the daytime, the driver might see the exit opening very bright when the driver is traveling inside the tunnel toward the tunnel exit. In the case of this phenomenon, the driver might see this exit opening looks like a "white hole". This phenomenon is opposite to the phenomenon at the tunnel entrance.
In case of the traffic volume increases and the distance between vehicles become shorter, the visibility of preceding cars that follow behind a large size car decreases under this phenomenon. The lighting of the tunnel exit is required to compensate for this decreasing visibility.

(1) Requirements

- Design traveling speed is more than 80 [Km/h]
- Outside luminance near Tunnel exit is more than 5,000 [cd/m^2]
- Tunnel length is more than 400 [m]

(2) Road surface luminance and Length

The vertical surface illuminance of the lighting for the Tunnel exit on the road height of 0.7 [m] shall be 12 [%] of the luminance near the tunnel exit. Thus, if the luminance is 5,000 [cd/m^2], the vertical surface illuminance is 600 [lx]. The length of the Tunnel exit zone is preferably 80 [m].
In case the Tunnel exit zone has an uphill gradient, it is possible to see a high luminance sky from inside the tunnel. The tunnel is structured so that sunlight is easy to insert into the tunnel. Therefore, even if the design traveling speed is less than 60 [Km/h], tunnel exit lighting should be installed.
Also, at the tunnel exit, placing trees and a sunshade louver to alleviate is recommended to avoid the sudden change of brightness due to natural light.

8.6.4 Lighting in Special structure part

Lighting in the Special structure part of the tunnel is the lighting which is mounted on the tunnel parting part, emergency parking area, pedestrian walkway, and evacuation passage.

(1) Lighting for the tunnel parting part

This lighting is required due to visual recognition of the existence of vehicles that are diverging / merging by illuminating the parting part.
The following points are important for this lighting.

- Identification of merging position by increasing luminance
- Properly arrange the lightings in consideration of the optical inductive effect

Those lightings are preferably set to 1.5 to 2.0 times the average road surface luminance of the basic illumination according to traffic volume.

(2) Lighting for the emergency parking area

This lighting is required due to visual recognition of the position of the emergency parking area from the running car.
The following points are important for this lighting.

- Identification of diverging/merging position by increasing luminance
- Identification of lighting position by changing the position and the light color for lightings

Those lightings are preferably ensured more than 50 [lx] the average road surface luminance of the lighting in the interior zones even if during nighttime dimming. In addition, those lightings shall be selected the lighting with good color rendering property in order to improve the visual performance of the evacuated car.

(3) Lighting for the pedestrian walkway

Those lightings are required to ensure the safety of pedestrians in the walkway.
Those lightings are preferably ensured more than 50 [lx] the average road surface luminance of the lighting in the interior zone even if during nighttime dimming. However, those lightings are not necessary if the required illuminance can be ensured by the lighting in the interior zone even during nighttime dimming.

(4) Lighting for the evacuation passage

This lighting is required to ensure evacuation and safety in case of emergency. Those lightings have preferably ensured the illuminance required for maintenance since those lightings are used not only in an emergency but also in maintenance management. Also, those lightings need to be lit even during a power failure. The required illuminance of this lighting is indicated by the chapter of tunnel emergency facility.

8.6.5 Lighting for Emergency

If the lights go out by a blackout during a tunnel run, the driver feels some danger. For example, if there is not enough space between the cars, the driver feels the danger of a rear-end collision. Also, in the case of face-to-face traffic with a narrow width, the driver feels the danger of colliding with oncoming vehicles.
In order to not give psychological anxiety to the driver due to such a dangerous situation, it is necessary to install lighting for the power failure. Lighting for Emergency for the power failure is composed of three kinds of methods.

- Method using uninterruptible power supply unit combining storage battery and a thyristor-inverter.
- Method using a reserve power source such as private power generation equipment.
- Built-in battery type system with power failure equipment.

8.6.6 Lighting for the connecting road

Lighting for the connecting road is required due to visual recognition of changing width near the tunnel entrance part or situation of the continuing road from tunnel exit part to the connecting road during the nighttime.

8.7 Lighting system

The lighting method of tunnel lighting is classified as shown below according to the position of lighting.

Table 8-7: Classification of lighting method

Lighting position / Lighting method	Lighting in the interior zone	Lighting for the Tunnel entrance
Symmetric lighting method	✔	✔
Counter beam lighting method	N/A	✔
Pro-beam lighting method	✔	✔

Source: Road tunnel (Lighting) technical standard and explanation; Japan Road Association 2009.

In addition, the lighting method of Tunnel entrance lighting is classified as shown in the table below according to the tunnel design traveling speed and the designed traffic volume.

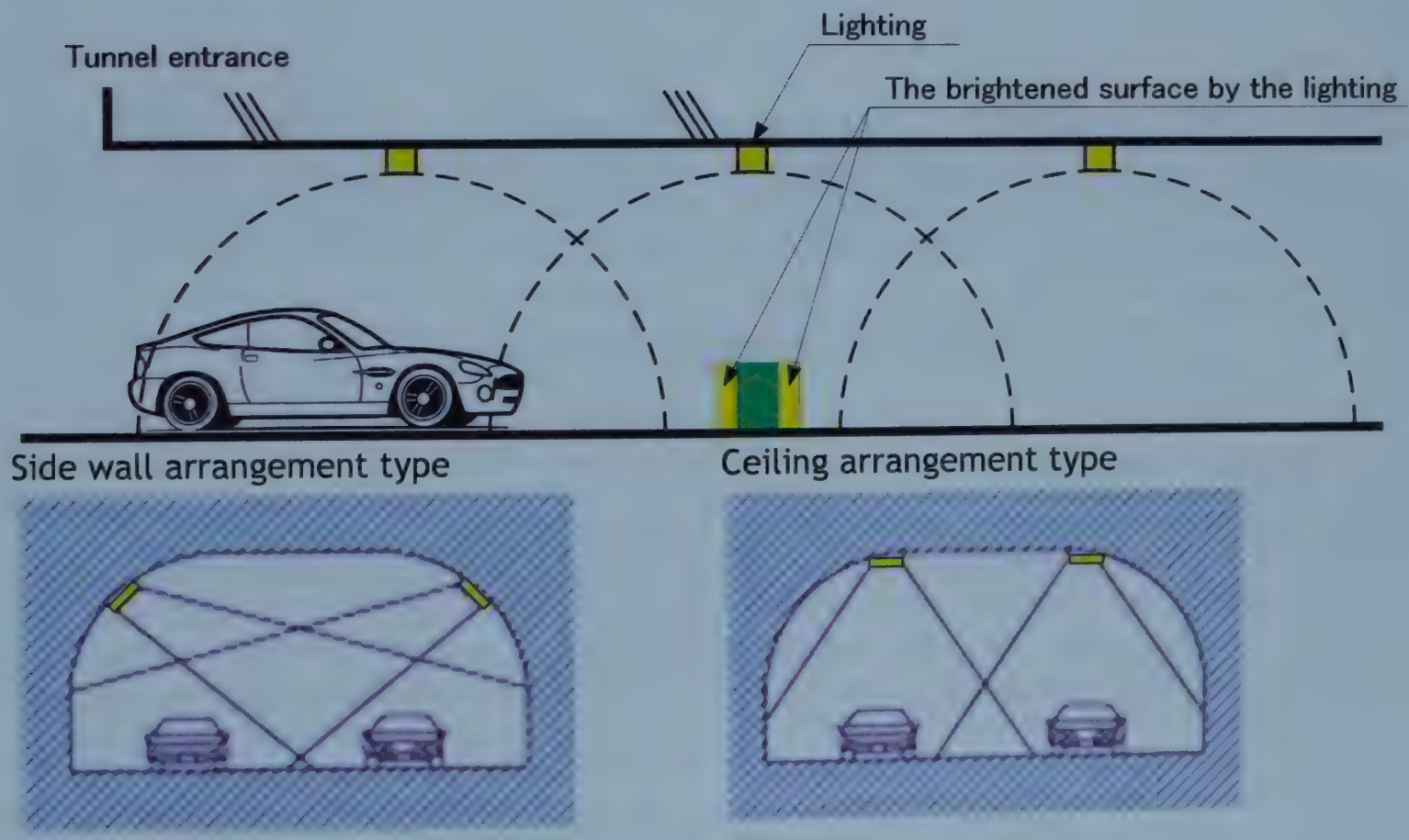
Table 8-8: Lighting method for the Tunnel entrance

		Design traveling speed		
		High	Middle	Low
Design Traffic volume	Many	Pro-beam lighting		Symmetric lighting
	Middle			
	Few	Counter beam	lighting	

Source: JICA Project Team.

8.7.1 Symmetric lighting method

The symmetric lighting method is the illumination system in which the light distribution to the vehicle on the road becomes symmetrical with respect to the road axis. It is a well-balanced lighting system that makes it easy to realize the visibility of vehicles and obstacles on the road and secure the required wall luminance.
Depending on the installation position of the lighting fixtures, it is classified into the following two methods.



Source: JICA Project Team.

Figure 8-11: Symmetric lighting method

(1) Side wall arrangement type

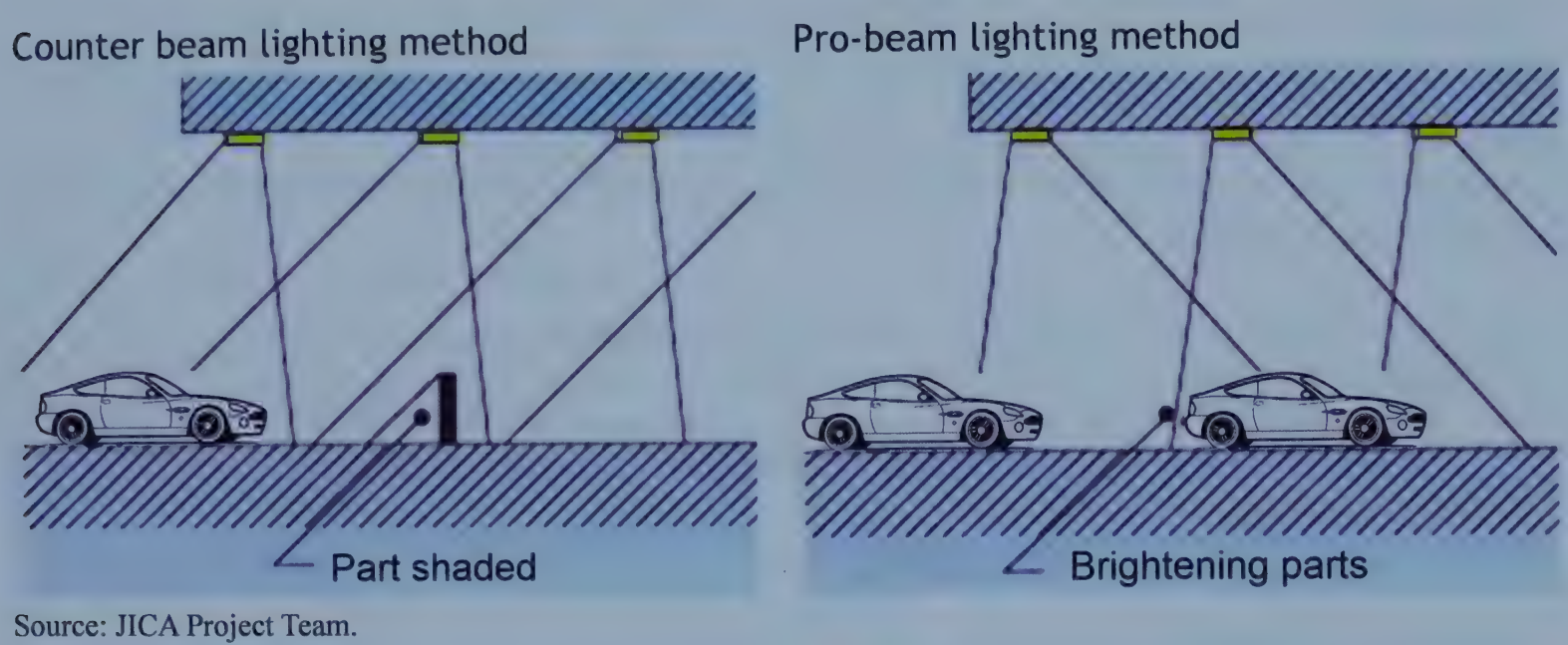
The method of installing lighting equipment on the ceiling side of the tunnel and irradiating it to the opposite lane.

(2) Ceiling arrangement type

The lighting system in which the lighting fixture is installed on the ceiling of tunnel and is irradiated directly below it. By positively irradiating the wall surface close to the installed lighting equipment, the wall surface luminance is increased and the viewing environment is improved.

8.7.2 Asymmetric lighting method

The asymmetric lighting method is the illumination scheme in which the light distribution to the vehicle on the road is symmetric with respect to the road axis. And, asymmetric lighting method composes of two methods in following, the Counter beam lighting method with light distribution opposite to the traffic direction (vehicle traveling direction) and the Pro-beam lighting method with light distribution in the traffic direction.



Source: JICA Project Team.

Figure 8-12: Asymmetric lighting method

(1) Counter beam lighting method

In the counter beam lighting method, these lightings are located on the opposite side of the driver's side of preceding cars so that the luminance ratio among the obstacle and the preceding car and the road surface might be high. Visibility of obstacles and preceding vehicles on the road will be higher by this method. Besides, even if the reflectance of obstacles is high, degradation of visibility is suppressed.
In the case of using the lighting of the tunnel entrance, low numerical values can be used for the setting value of Luminance in the Threshold zone, as compared with the Symmetric lighting method. However, when the natural light is strongly injected, the vertical illuminance of the back surface of the obstacle decreases, so that the visibility drops compared to the Symmetric lighting method.

(2) Pro-beam lighting method

The pro-beam lighting method is an illumination scheme that distributes light in the traffic direction.
In case that the illuminance on the vertical surface of the obstacle backside (natural light is strongly injected like a sunrise or sunset rises), the Pro-beam lighting method increases visibility.
Therefore, the Pro-beam lighting method is used at the tunnel entrance and exit where the importance of visibility of obstacles and preceding cars is regarded as important

when the design traveling speed is high, the traffic volume is high, or the luminance in the access zone is high.

8.8 Night lighting

Night lighting in mountain road tunnels must be installed in order to ensure the safety of road users.
Basically, lighting inside the tunnel at night should satisfy the following specifications.

- The average road surface luminance: 1[cd/m^2] and more,
- The uniformity ratio of illuminance in the tunnel at nighttime should be fulfilling the same requirements as the daytime lighting.

If the tunnel is on a section of a road continuously illuminated, the quality of the lighting inside the tunnel should be at least equal to the level, uniformities, and glare of the illuminations for this road.

8.9 Operation and control

Operational control of tunnel lighting must ensure traffic safety with efficiency and economical. In other words, by operating the tunnel lighting, each lighting facility is controlled illuminance (by dimming control, etc.) according to luminance in the access zone and traffic volume.
The dimming of the road surface luminance controls the luminance of the lighting and the lighting time of the lighting depending on the tunnel, the situation around the tunnel, the traffic volume, and the safety. However, due to the dimming of road surface luminance, traffic safety should not be reduced.

8.9.1 Operation for the lighting in the interior zone

The lighting in the interior zone can be reduced the road surface luminance by dimming control if the traffic volume decreases and the tunnel internal vision environment in the nighttime could improve. This dimming control can be set up the value shown in Table 8-1 "Average road surface luminance for lighting in the interior zone" to 1/2 (nighttime) or 1/4 (midnight). However, the lowest road surface luminance is 0.7 [cd/m2].
The light reduction of all lightings is eminently suitable for the dimming control for the lighting in the interior zone than the partial lights-out.
In the case of partial lights-out, the total uniformity ratio of illuminance (U_0) for road surface luminance should be as high as possible. Moreover, the road surface luminance of lighting in the interior zone shall be controlled by the luminance in the access zone when the road surface luminance of lighting in the interior zone is 1/2 lighting from all lighting. And, the road surface luminance of lighting in the interior zone shall be controlled by the timer when the road surface luminance of lighting in the interior zone is 1/4 lighting from1/2 lighting.

8.9.2 Operation for the lighting in the tunnel entrance and exit

The road surface luminance in the tunnel entrance shall be controlled to the required road surface luminance according to the luminance in the access zone. And, this dimming control of the lighting in the tunnel entrance is in four stages (high design traveling speed) or two stages (low design traveling speed) according to the road surface luminance in the tunnel entrance. The road surface luminance in the tunnel entrance is controlled so as to be the road surface luminance ratio shown in the following table according to the luminance in the access zone.
The road surface luminance in the exit zone shall be controlled by the same as to the tunnel entrance.

Table 8-9: Dimming control for lighting in the tunnel entrance (five stages)

Ratio to the luminance in the access zone setting value	Road surface luminance ratio
75 [%] and more	100 [%]
50 [%] to 75 [%]	75 [%] and more
25 [%] to 50 [%]	50 [%] and more
5 [%] to 25 [%]	25 [%] and more
5 [%] and less	0 [%]

Source: Road tunnel (Lighting) technical standard and explanation; Japan Road Association 2009.

Table 8-10: Dimming control for lighting in the tunnel entrance (three stages)

Ratio to the luminance in the access zone setting value	Road surface luminance ratio
50 [%] and more	100 [%]
5 [%] to 50 [%]	50 [%] to 100 [%]
5 [%] and less	0 [%]

Source: Road tunnel (Lighting) technical standard and explanation; Japan Road Association 2009.

8.10 Design

Lighting design should determine the source of lighting, the light distribution of the lamp, and the arrangement of the lighting equipment according to the provisions of chapters 2.6 to 2.9.

8.10.1 Requirement

(1) Specified value and recommended value

The prescribed values shown in previous chapters are values specified as a performance index in tunnel lighting. On the other hand, the recommended values are values that desirably satisfy those values.

Since the lighting facilities of the tunnel lighting may be related to each other depending on their installation positions, the continuity of the lighting facilities must be taken into consideration and comprehensively studied.

The relationship between each lighting facility and the corresponding performance index (specified value) and recommended value is shown in the table below.

Table 8-11: Relationship between each lighting facility and the corresponding performance index (specified value) and recommended value

		Interior zone	Entrance zone	Exit zone	Special structure part	Connecting road
Performance index (specified value)	Average road surface luminance	✓	✓			
	Total uniformity ratio of illuminance	✓				
	Increase of relative threshold	✓				
	Inductive	□				
Recommended value	Average road surface illuminance				✓	△ *5
	Uniformity ratio of illuminance along lane axis	△ *1				
	Increment of relative threshold		△ *2			
	Wall surface luminance	✓	△ *3			
	Vertical illuminance		△ *4	✓		

Source: Road tunnel (Lighting) technical standard and explanation; Japan Road Association 2009.

Legend; ✓: For designing, △: For designing if it's required, □; No need for evaluation

Note;
*1; Uniformity ratio of illuminance along lane axis shall be subject for designing if the average road surface luminance had been decreased by the design traveling speed and the traffic volume.
*2; Increase of relative threshold shall be subject to designing in the case of the counter beam lighting method. However, it can be omitted when the lighting is an adaptable specification for the counter beam lighting method.
*3; Wall surface luminance can be omitted if the light axis direction / light distribution of the lighting is equivalent to the light axis direction / light distribution of the lighting used for the interior zone.
*4; Vertical illuminance shall be subject for designing in the case of the pro-beam lighting method.
*5; Average road surface illuminance shall be subject to designing in case of a plurality of lightings are installed.

(2) Calculation method of lighting design

There are two ways to calculate lighting design;
- Lumen method,
- Point method.

The point method is classified by "illuminance calculation by the point method" and "luminance calculation by the point method."
The lumen method is a calculation method for obtaining the average illuminance of the light-emitting surface which becomes brighter by the luminous flux incident on a certain surface from the lighting.

The point method is a calculation formula for obtaining the following for an arbitrary point in the surface to be illuminated.

- The luminous intensity incident from the lighting
- The illuminance and luminance, depending on the distance and angle between the light source and the arbitrary point

The calculation method of the lighting design for each design object is shown in the table below.

Table 8-12: Calculation method of lighting design

		Lumen method	Point method
Performance index (specified value)	Average road surface luminance	✓	
	Total uniformity ratio of illuminance		✓
	Increase of relative threshold		✓
	Inductive	□	□
Recommended value	Average road surface illuminance	✓	
	Uniformity ratio of illuminance		✓
	Uniformity ratio of illuminance along lane axis		✓
	Increment of relative threshold		✓
	Wall surface luminance	✓	
	Vertical illuminance		✓

Source: JICA Project Team.

Legend; ✓: For designing, □; No need for evaluation

8.10.2 Average road surface luminance calculation by the lumen method

The lumen method is a commonly used calculation method and is used for calculating the average illuminance, the instrument spacing, and the number of lightings.

(1) The calculation for average road surface illuminance (E_r)

The average road surface illuminance (E_r) can be obtained by the following equation.

$$E_r = \frac{F \cdot U \cdot M \cdot N}{S \cdot W} \ [lx]$$

F	:	Lighting speed of the source	[lm]
U	:	Utilization factor	N/A
M	:	Maintenance factor	N/A
N	:	Arrangement of lighting	N=2: face to face arrangement, N=1: staggered, central, one side arrangement N/A
S	:	Spacing for lights	[m¥
W	:	Wide of road	[m]

(2) Calculation for average road surface luminance (L_r)

The average road surface luminance (L_r) can be obtained by the following equation.

$L_r = \frac{F \cdot U \cdot M \cdot N}{S \cdot W \cdot K}$ [lx/cd /m²]

K : Average illuminance conversion factor [lx/cd/m²]

(3) Average illuminance conversion factor (K)

The average illuminance conversion factor is a coefficient for converting the average road surface luminance to the average road surface illuminance, and it varies depending on the light distribution and arrangement of the lamp other than the type of the road surface. The average illuminance conversion factor is shown in the table below according to past results and research.
Therefore, the average illuminance conversion factor for draining pavement and light-colored pavement, lighting with low height pillar, low-level lighting of balustrade luminaire shall be required to be used appropriately by further research and research.

Table 8-13: Average illuminance conversion factor (K)

	The type of the road surface	
	Concrete	Asphalt
Road lighting	10 [lx/cd/m²]	15 [lx/cd/m²]
Tunnel lighting <Symmetric lighting method>	13 [lx/cd/m²]	18 [lx/cd/m²]
Tunnel lighting <Asymmetric lighting method> *1	9 [lx/cd/m²]	

Source: Road tunnel (Lighting) technical standard and explanation; Japan Road Association 2009.
Note;
*1; In this case, the asymmetric illumination method is a counter beam illumination method, and these lightings are disposed of in the width of a roadway or near the boundary between a roadway and a shoulder. The average illuminance conversion factor should be required to investigate when another asymmetric illumination method except the counter beam illumination method is adopted.

(4) Utilization factor

The utilization factor is calculated by the curve of the utilization factor. For the mutual reflection component of ceiling/wall/road surface which are consisted of a combination of the following;

- The lighting source,
- The lighting equipment the road width,
- The light position,
- The mounting height of the lighting equipment,
- The mounting angle of the lighting equipment

(5) Maintenance factor

In the lighting facility, the road surface luminance/road surface illuminance decreases from the initial value due to a decrease in the luminous flux of the lighting source, contamination of the lighting fixture, and a decrease in wall reflectivity. The maintenance rate is a numerical value representing a decrease in illuminance due to these factors and is a coefficient to be expected in design. Recommended values of maintenance rate are shown in the table below.

Table 8-14: Maintenance factor

	Maintenance factor
Road lighting	0.65~0.75
Tunnel lighting	0.50~0.75

Source: Road tunnel (Lighting) technical standard and explanation; Japan Road Association 2009.

Factors that constitute the maintenance rate are the road structure, the traffic situation, the replacement time of the light source, the replacement method of the light source, and the cleaning frequency of the lighting. Therefore, the set maintenance rate should adopt appropriate values in consideration of the following;

- Traffic volume,
- Vehicle composition,
- Road surrounding situation and maintenance

8.10.3 Average wall surface luminance calculation by the lumen method

(1) The calculation for average wall surface luminance

The average wall surface luminance (L_w) is calculated to obtain the luminance ratio between the wall surface and the road surface for the evaluation of the visual environment. And, the calculation of the average wall surface luminance (L_w) uses the following equation with the wall surface as a perfect diffusion plane.

$$L_w = \frac{\rho_w \cdot E_w}{100\pi} = \frac{\rho_w}{100\pi} \cdot \frac{F \cdot U_w \cdot M \cdot N}{S \cdot H_w} \ [cd/m^2]$$

ρ_w : Reflection ratio of the wall [%]
E_w : Average illuminance of the wall [lx]
F : The light flux per lighting fixture [lm]
U_w : Lighting rate of the wall surface from the road surface to Hm (including mutual reflection).
M : Maintenance factor
N : Arrangement factor (1: One side, Center, Zigzag. 2: Orientation)
S : Lighting spacing [m]
H_w : Height of calculating wall [m]

(2) The calculation for luminance ratio between the road surface and the wall surface

The luminance ratio between the road surface and the wall surface is expressed by the following formula.

$$\frac{L_w}{L_r}$$

L_w : Average wall surface luminance [cd/m^2]
L_r : Average road surface luminance [cd/m^2]

The luminance ratio between the road surface and the wall surface can also be calculated from the illumination rate (U_r) of the road surface, the illumination rate (U_w) of the wall surface, the area of the light surface (road surface and wall surface) and the reflectance.

8.10.4 Illuminance calculation by the point method

The point method is a calculation formula for obtaining the following for an arbitrary point in the surface to be illuminated for preparing an illuminance distribution map and calculating illuminance uniformity.

- The luminous intensity incident from the lighting
- The illuminance and luminance, depending on the distance and angle between the light source and the arbitrary point

Normal illuminance (E_n) and horizontal plane illuminance (E_n) calculated by the point method shown in the figure below are shown in the following formulas.

$$E_n = \frac{I_\theta}{l^2}\ [lx]$$

$$E_h = E_n \cdot cos\theta\ [lx]$$

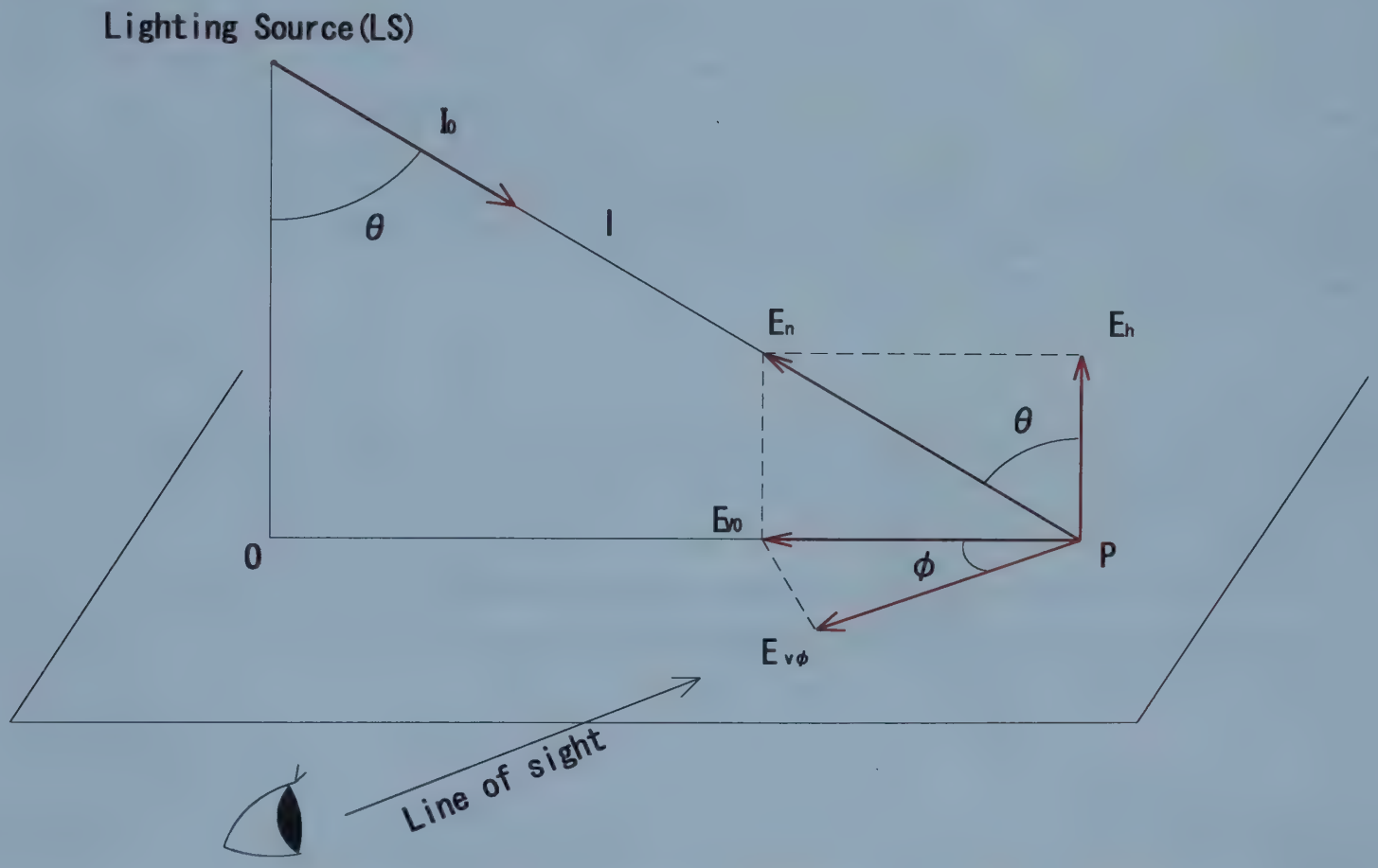


Figure 8-13: Illuminance calculation of the point method

I_θ	:	Luminous intensity from the light source (lighting fixture) to point P direction	[cd]
L	:	Distance from the light source (lighting fixture) to point P direction	[m]
θ	:	The angle between the normal from the point P to the light source and the perpendicular	[deg.]
E_n	:	Normal illuminance at point P	[lx]
E_h	:	Horizontal illuminance at point P	[lx]

In addition, the vertical surface illuminance (E_{vo}) of the point P in the direction of the point O directly under the light source and the vertical surface illuminance ($E_{v\varphi}$) in the horizontal angle φ are expressed by the following equations.

$$E_{vo} = E_n \cdot \sin\theta\ [lx]$$

$$E_{v\varnothing} = E_n \cdot \sin\theta \cdot \cos\varnothing\ [lx]$$

(1) Calculation of illuminance uniformity (U_{OE})

The illuminance uniformity (U_{OE}) is calculated by dividing the minimum illuminance of the target range by the average road surface illuminance.

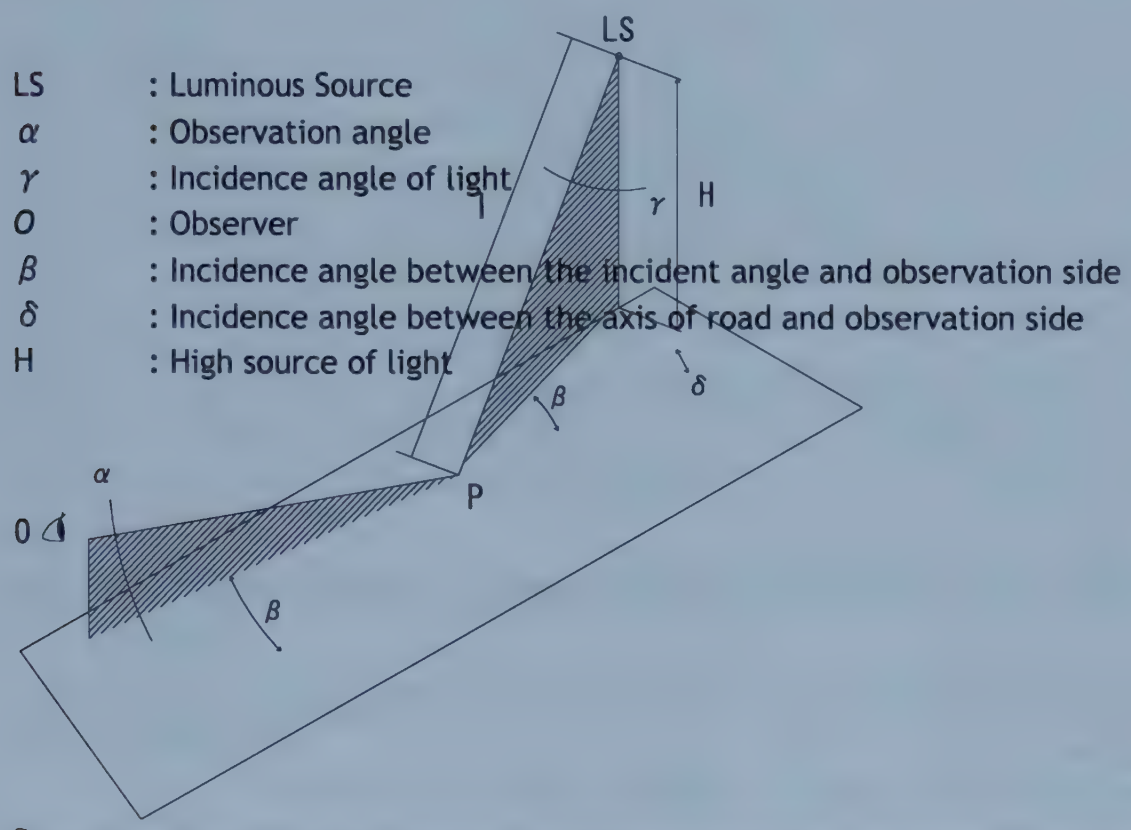
$$U_{OE} = \frac{E_{min}}{E_r}$$

E_{min} : Minimum illuminance of the target range [lx]
E_r : Average road surface illuminance of the target range [lx]

8.10.5 Luminance calculation by the point method

In the illuminance calculation by point method, in order to calculate the illuminance distribution and the illuminance uniformity, the illuminance of a specified point within a predetermined calculation area is calculated.
And, luminance calculation of point by point method can calculate the luminance of each point by using the standard road surface reflection properties which are recommended by CIE. For this calculation, 'CIE Pub.No.30.2 "Calculation and Measurement of Luminance in Road Lighting 2nd"' can be referred to.
In general, the luminance multiplies the luminance coefficient of the position that wants to calculate by road surface illumination. The luminance coefficient (q) is expressed by three points (viewpoint, road surface, light source) of positions. The formula and the figure for explanation are shown below.

LS : Luminous Source
α : Observation angle
γ : Incidence angle of light
O : Observer
β : Incidence angle between the incident angle and observation side
δ : Incidence angle between the axis of road and observation side
H : High source of light



Source: Road tunnel (Lighting) technical standard and explanation; Japan Road Association 2009.

Figure 8-14: Figure for luminance calculation of point by point method

$$q = \frac{L}{E}$$

L : Luminance [cd/m²]
E : Illuminance [lx]

The illumination of point P in the above figure is calculated like a formula below by using the point by point method.

$$E = \frac{I}{l^2} cos\gamma \ [lx]$$

I	:	The brightness to the position P from the lighting source (LS)	[cd]
l	:	The length to the position P from the lighting source (LS)	[m]
γ	:	The included angle between the perpendicular line and normal line from point P to lighting source (LS)	[deg.]

And, '$l^2 = H^2 / \cos^2(\gamma)$' because of '$\cos(\gamma) = H / l$'
Therefore, the luminance (L) of point P in the above figure is calculated like the formula below.

$$L = q \cdot \frac{I}{H^2/\cos^2\gamma} \cdot \cos\gamma = \frac{I \cdot q \cdot \cos^3\gamma}{H^2} = \frac{I \cdot r}{H^2} \ [cd/m^2]$$

H	:	Height of lighting equipment	[m]
r	:	Luminance conversion factor	[q · cos³ (γ)]

The list for luminance conversion factor (r) is standardized by CIE.

(1) Total uniformity ratio (U_o)

The uniformity ratio of luminance is composed of the total uniformity ratio and uniformity ratio of the lane axis. The total uniformity ratio is shown in the following formula.

$$U_o = \frac{L_{min}}{L'_r}$$

L_{min}	:	Minimum partial luminance within the range of object	[cd/m²]
L'_r	:	Average road surface luminance by point method	[cd/m²]

(2) Uniformity ratio of lane axis (U_l)

The uniformity ratio of the lane axis is shown in the following formula.

$$U_l = \frac{L_{min}[l]}{L_{max}[l]}$$

L_{min} [l]	:	Minimum partial luminance on the centre line of each lane	[cd/m²]
L_{max} [l]	:	Maximum partial luminance on the centre line of each lane	[cd/m²]

8.10.6 The relative threshold increase calculation

The evaluation method for discomfort glare and glare which deteriorates a visual function should refer CIE Pub.No.31 "Glare and Uniformity in Road Lighting Installations (1976)". These glares which deteriorate a visual function (a relative threshold increase [TI]) are shown in the following calculation method.

$$In\ case\ of\ L_r \leqq 5\ cd/m^2, \quad TI = 65 \cdot \frac{L_v}{L_r^{0.8}} \ [\%]$$

$$In\ case\ of\ L_r > 5\ cd/m^2, \quad TI = 95 \cdot \frac{L_v}{L_r^{1.05}} \ [\%]$$

L_r	:	Average road surface luminance	[cd/m²]
L_v	:	Equivalent veiling luminance by the lighting in the driver's view	[cd/m²]

As for these formulas, the maintenance factor is calculated as one (1).

Equivalent veiling luminance (L_v) is shown the following formula.

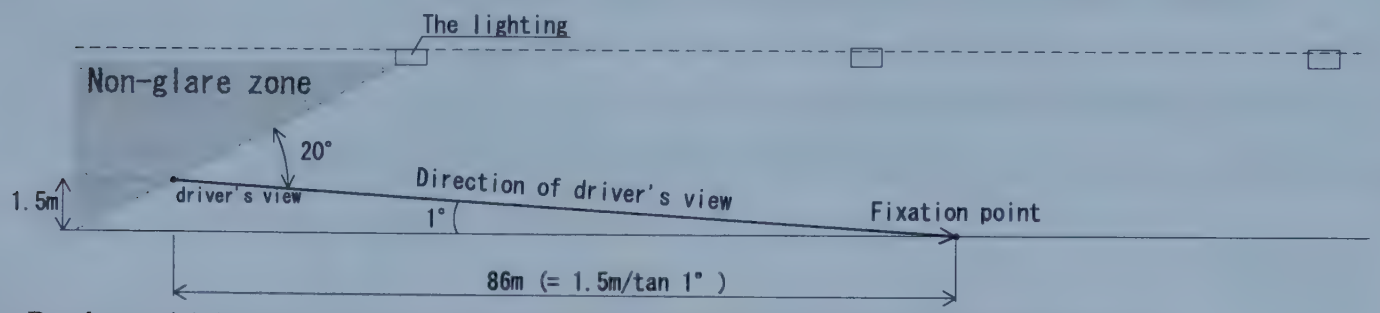
$$L_V = 10 \cdot \sum_{i=1}^{n} \frac{E_{vi}}{\theta_i^2} \quad [cd/m^2]$$

E_{vi}	:	Illuminance on the perpendicular side in a sight line by glare source (i)	[lx]
θ_i	:	An included angle between a sightline and glare source (i)	[deg.]
I		The number of the lighting	[units]

The calculation condition is shown in the following.

(a) The position of the driver's view is equal to 1.5[m] from the centre line of each lane.
(b) The angle of elevation which is set to the first lighting from the driver's sightline is twenty degrees.
(c) The maximum equivalent veiling luminance along the lane axis shall be calculated with these requirements.
(d) The fixation point is set to the point forward which makes intersected driver's sight line is one degree.
(e) The fixation point becomes the point of the angle of depression one (1) degree from the driver's view.
(f) The range of θ_i is 1.5 from 60 degrees.

The position of the driver's view is set to the position shown in the figure below. This position is a position in which the maximum value of the equivalent veiling luminance is given (momentary position in which lighting is shaded in the upper side of the windshield).



Source: Road tunnel (Lighting) technical standard and explanation; Japan Road Association 2009.

Figure 8-15: The position of driver's view for calculating a relative threshold increase

CHAPTER 9 Tunnel Electric Room

9.1 Requirement

The tunnel electric room is necessary to set up the following equipment.

- Equipment and wiring cable for monitoring and controlling the facilities located inside the tunnel,
- Equipment and wiring cable for receiving the power from the power corporation,
- Sub-station for distributing the power to the facilities,
- Emergency power generator for power failure,
- Fuel tank for generator
- Pump and pipe for feed water and drainage,
- Equipment for snow removal,
- Manhole

Therefore, the tunnel electric room should be located close to the tunnel portal, in order to shorten the layout length of wiring cables and piping. Also, in addition to securing space according to the size of the tunnel, it is necessary to consider the space for snow removal, the size of the water tank and fuel tank as necessary.

On the other hand, when the number of facilities is less, the tunnel electric room may not be required due to high-cost factor of constructing a tunnel electric room.

9.2 Installation place

The installation place for the tunnel electric room must be considered as follows.

- Installation places such as an electrostatic precipitator, a supply, an exhaust blower, ventilation equipment.
- In the case of placing an electric room in a tunnel, the groundwork is adjusted to the progress of civil engineering work.

9.3 Load capacity

A load capacity of a tunnel facility will vary depending on the specification, the scale of a facility, and operation method. Total load capacity is calculated by accumulating all the required capacity of equipment respectively. Total load capacity shall also be the supply power capability of the substation. The scale and design of the substation must be decided by calculating this load capacity.

9.4 Power receiving method

In the power receiving method, the optimal number of power receiving circuits (one circuit or two circuits) should be chosen in consideration of load capacity, power transmission distance, voltage drop, lighting equipment, power distribution system of transmission, and distribution business operators. For tunnels with disaster-prevention class AA or higher, consider high-voltage distribution lines to be redundant in order to

improve the reliability of the power supply. The table below shows an application example of the power receiving method.

Table 9-1: Power receiving method

Tunnel facility conditions		Power receiving method
Mechanical ventilation	Two number or more than electric rooms	Power receiving; One or two circuits, and a standby generator facility
	One electric room	Power receiving; One circuit, and a standby generator facility
Natural ventilation	Disaster-prevention class A or higher	Power receiving; One circuit, and a standby generator facility
	Disaster-prevention class B or lower	Power receiving; One circuit

Source: Design notes (Tunnel); Nippon Expressway Company Limited 2015.

Note; for disaster prevention class, refer to Chapter of Tunnel Maintenance Management.

9.5 Power distribution line

The design of the power distribution line should consider the following points.

- In case that the length of the high voltage cable wiring is long, an isolation transformer and a grounding transformer should be installed for separating the grounding system so that a charging current to the ground of the high voltage cable wiring doesn't become big.
- The voltage drop of the wiring cable should be in accordance with the separately specified standard.

9.6 Wiring

The wiring design should consider the following points.

- The wiring in the exposed pipe and the cable rack wiring of the distribution line used for lighting at the time of power failure should be specified as fireproof specifications.
- Fixed methods of cable for the wiring include synthetic resin pipes, metal pipes, cable racks, cable ducts, trenches, and concrete troughs. However, this construction planning should be made into consideration with the cross-sectional shape of the laying place and maintenance.

9.7 Ground

Grounding design shall be in compliance with the separately prescribed law. For grounding of lighting, CCTV, ventilation, and disaster prevention facilities use a common grounding wire laying method.

9.8 Others

Other items which required attention are shown below.

- Voltage change measures (flicker jab measures) and harmonics measures should be considered.
- The capacity of the fuel tank of the power generation facility should be the capacity that can operate continuously for at least 24 hours.